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This report presents a review and discussion of several procedures for designing structures to resist earthquake shaking. The cost of strengthening structures is presented, and the damage expected from earthquakes is discussed. A least-total-cost analysis is presented which leads to optimal design acceleration levels. Navy economic analysis techniques were used to evaluate present worth of damage and cost. Results are presented in terms of least-cost design levels as a function of site earthquake acceleration levels having a probability of not being exceeded in 50 years. Also given is the marginal cost per life saved for the present Naval Facilities Engineering Command design level over the optimal level. Results indicate that a reduction in design level is possible.

CONTENTS

INTRODUCTION

The seismic design of a structure can be a complex task. Several alternative design techniques are in use. These vary from a relatively simple static assignment of shear forces to a full complete nonlinear dynamic analysis. The Naval Facilities Engineering Command (NAVFAC) guidelines specify that a site seismicity study be performed for critical buildings in lieu of the lateral load coefficients. Further, the level of design earthquake is suggested as the ground acceleration having an 80% chance of not being exceeded in 50 years.

An automated procedure has been developed at the Civil Engineering Laboratory (CEL), as part of an on-going study, to perform a seismic analysis using available historic and geologic data. The objective of the seismicity study is to determine the probability of occurrence of ranges of accelerations at the site. To do this, the site coordinates and the study bounds are specified in terms of latitude and longitude. A regional study is first performed in which all of the historic epicenters are used with an attenuation relationship to compute site acceleration for all historic earthquakes. A regression analysis is then performed to obtain regional recurrence coefficients, and a map of epicenters is plotted. The regional recurrence can be used to compute the probability of site acceleration for randomly located events in the study area. Such a condition is used when individual faults are not known well enough to be specified.

Where individual fault areas can be specified, individual subsets of the historic data are used in conjunction with geologic data to determine fault recurrence coefficients, which are used to compute the probability of site acceleration from individual fault sources. The total risk is determined for all faults specified. Confidence bounds are given on the site acceleration as a function of probability of not being exceeded.

The structural design engineer may use either response spectra or time history techniques in the analysis of a structure. The California Institute of Technology data base of recorded accelerograms has been obtained by CEL and installed for NAVFAC use. A program was prepared to search the record of accelerograms, given a desired magnitude event, epicenter-site distance, acceleration level, and soil condition to determine the closest matching records. Another program prepared by CEL takes selected response spectra, scales them, and computes mean and standard deviation spectra and maximum envelope spectra. These spectra are plotted either in tripartite form or in semilog form. A program was prepared to scale, plot, and punch time history accelerograms for use as input to dynamic finite element programs.

References 1 and 2 were prepared to document the programs and procedures. A case study of the San Diego area is included in Reference 2. Reference 3 was prepared to document a typical case study in the Los Angeles area. Results compare favorably with results by others. Reference 4 was prepared to study a site where faulting could not be identified readily and regional analysis techniques had to be used. The following list summarizes programs written at CEL as part of this task.

Program RECUR	Computes probability of site acceleration given historic and geologic data
Program OPTIREC	Optimally selects available accelerogram to match site condition specified
Program RESPLOT	Selects, scales, determines average, etc., of response spectra and plot results
Program TIMHIS	Selects, scales, and plots time history accelerogram records

The following programs were obtained from other sources to support this work.

Program CH42	NOAA earthquake epicenter data and program to search tape (NOAA)
Program SIMQKE	Generates set of accelerograms given input response spectra (MIT)
Program SPECEQ	Computes response spectra from accelerogram

All of the programs are installed on the Control Data System and are available for NAVFAC and Engineering Field Division (EFD) use. The programs have already been used by EFDs.

A standardized procedure has been developed to determine probability of site acceleration using both historic and geologic data. By use of this information, a site-matched acceleration response spectra or time history can be created. Reference 5 focused on the use of response spectra and time history techniques, specifically looking at the computation of response when a structure is present, (particularly in soil-structure interaction). That report demonstrated the sensitivity of structural response to input data.

Previous CEL reports demonstrate a suggested approach for computing the probability of seismic loading at a site. The total plant replacement value of Naval installations in Uniform Building Code (UBC) seismic zones 3 and 4 is approximately $21 billion. Current NAVFAC criteria specify a 225-year return time earthquake (80% probability of not being exceeded in 50 years) as the basis for seismic design. The selection of this level was a NAVFAC judgment decision. This study will attempt to develop risk-optimized procedures for Navy structures to minimize damage and repair costs in relation to initial construction cost. A damage function relating expected damage as a function of imposed acceleration level can be constructed. This may be combined with a function relating cost of construction as a function of design acceleration level for use with the actual probability of site acceleration. Total cost may be related to design earthquake return time. The intent is to focus on the seismic design procedure, the earthquake design return period, and the level of structural performance in an attempt to produce the safest, least-cost structure.

It is important to note that the probability of site acceleration used in this study is based on the total risk to the site from <u>all</u> of the faults in the area. As such, it represents a more rigorous exact estimation of the actual risk. Less rigorous studies by others attempt to design for the most significant fault, ignoring all others. This is

a simplification perhaps useful for design; however, it understates the risk in most situations where there are several faults of almost equal seismic capability.

PERCEPTION OF THE SEISMIC PROBLEM

Perhaps the first recorded building code is contained in the Code of Hammurabi (Ref 6). Although specific design requirements are not provided, penalties are given for functional failure:

> "A. If a builder build a house for a man and do not make its construction firm and the house which he has built collapse and cause the death of the owner of the house - that builder shall be put to death.
>
> B. If it cause the death of the son of the owner of the house - they shall put to death a son of that builder.
>
> C. If it cause the death of a slave of the owner of the house - he shall give to the owner of the house a slave of equal value.
>
> D. If it destroy property, he shall restore whatever it destroyed, and because he did not make the house which he built firm and it collapsed, he shall rebuild the house which collapsed at his own expense.
>
> E. If a builder build a house for a man and do not make its construction meet the requirements and a wall fall in, that builder shall strengthen the wall at his own expense."

It is obvious that this code would have a conservative influence on the designer.

The 1906 earthquake caused major damage in San Francisco, yet it was not until 1948 that comprehensive lateral force design criteria specifically referring to seismic forces was adopted there. In 1969 a parapet ordinance was passed. Although the mayor signed the ordinance he did not support it publicly on the grounds that the doomsday mentality would deprive San Francisco of its exterior beauty (Ref 7).

In 1976 less than 5% of California home owners were covered by earthquake insurance. Historically, the insurance industry has not promoted the sale of earthquake insurance based on the concern of large losses should a severe disaster occur. The problem of high losses is caused by the phenomenon of adverse selection, whereby only people in hazard-prone areas wish to buy insurance coverage, thus necessitating high rates while concentrating coverage on small risk-prone areas. Rates are a function of the risk zone in which they are located. Table 1 illustrates rates for California, which is divided into three zones (Ref 8). Depending upon the rate of return of invested money, the insurance industry break-even period for frame structures is planned for events with over 50-year return times.

CURRENT DESIGN PROCEDURES

Uniform Building Code (UBC) 1979 Edition

The seismic design is specified in Section 2312 of the 1979 edition of the UBC (Ref 9). The minimum total lateral force or total one-direction base shear is specified by

$$V = Z\,I\,K\,C\,S\,W$$

where Z = numerical coefficient, dependent on the zone-specified map of the United States

Zone 1	$Z = 3/16$
Zone 2	$Z = 3/8$
Zone 3	$Z = 3/4$
Zone 4	$Z = 1$

C = based on natural period of structure; the period is based either on an analysis or an empirical equation

S = site structure interaction coefficient based on period of structure and period of soil

I = occupancy importance factor; varies from 1 to 1.5

K = numerical coefficient based on building type and bracing

W = total dead load

The procedure established is a working stress design approach. The total base shear is determined and then applied to the floor levels by a distribution equation using the weights and heights of the individual floors. The building is to be designed to resist overturning. Lateral story drift relative to an adjacent story shall not exceed 0.005 times the story height; the displacement calculated by application of the required lateral forces shall be multiplied by 1.0/K to obtain the drift, but not less than 1.0. Base shears are assumed to act nonconcurrently in the direction of each major axis of the structure. Torsion is considered directly by increasing shears; irregular structures require a dynamic analysis. Provisions are made for design of bearing and nonbearing interior and exterior walls. Roof and floor diaphragms must be designed for the lateral force. All allowable stresses and soil-bearing values may be increased by one-third for earthquake forces. Ultimate strength procedures are specified for concrete design. Load factors are specified. Provisions allow for a dynamic analysis. The design philosophy provides for a structure to remain essentially elastic under a low to moderate level of shaking prescribed for the zone. For ordinary frame construction in zone 4, the level of shaking represents an approximate 70-year return time for ordinary structures based on CEL site studies (although this varies considerably by location and distance from the fault system governing the motion). Although inelastic behavior is expected to occur under moderate to strong earthquakes, no provisions are prescribed for collapse level designs.

Figure 1 (from Ref 10) gives approximate damage estimates for design by UBC.

Structural Engineer Association of California (SEAOC) Code of 1975

The basic approach to lateral force computation is similar to the UBC presented in Reference 10. Lateral force on the elements of structures, such as parapets, partitions, and diaphragms, use different coefficients. The UBC contains modifications for zones 1 and 2 not found in the SEAOC code (Ref 11) and procedures for design of floor diaphragms are different. The UBC requires evaluation of equipment required to be functional after an earthquake for story drifts of 2.0/K times the normal allowable. As with the UBC the approach taken is an elastic-type design with the understanding that inelastic behavior may result under probable earthquake exposure. The philosophy limits stress and drift under a service-type load. SEAOC contains certain limitations on concrete shear walls and braced frames not found in the UBC. Collapse load levels are not prescribed.

ATC-2 by Applied Technology Council

The ATC-2 study (Ref 12) evaluated the response spectrum approach to the seismic design of buildings. The procedure requires a site seismicity study to establish a design level acceleration and a collapse level acceleration. The approach is to prevent significant damage during an earthquake with moderate intensity at the site and to prevent collapse during an earthquake of major intensity at the site. A damage threshold spectrum is established for a level of ground shaking with a return time of about 100 years. A collapse threshold spectrum is established based on maximum event and has a return period of about 660 years. A spectral shape is established using procedures developed by Newmark and Hall (Ref 13). An inelastic spectrum is established for design and collapse levels. Ductilities and damping are as specified in Table 2. Modal analysis techniques are specified for analysis of the structure. Modes are combined by square root of the sum of the squares of the individual modes. The strength capacity must be sufficiently high to prevent significant structural damage due to the damage threshold earthquake. The element yield must provide ductile deformation capacity. As noted in Table 2 inelastic behavior is specified for both design and collapse levels. Load factors U for member design are expressed as:

$$U = D + L + E$$

$$U = (2/3)\,D - E$$

where D = dead

L = live

E = earthquake

Since large deformations are possible, P - Δ effects from frame sidesway must be considered. Two-directional seismic effects are combined by prescribing 100% of the force in the main direction and 30% of the orthogonal force applied in the orthogonal direction. Each level of spectrum considers both strength and displacement. Story drifts are set at four times UBC levels for the collapse level.

ATC-3, Applied Technology Council Provision for Seismic Regulations for Buildings

A study (Ref 14) was performed to prepare tentative design provisions applicable to earthquake areas of the United States. The philosophy in establishing design levels was to resist minor earthquakes without damage, to resist moderate earthquakes without significant structural damage but with some structural damage, and to resist major earthquakes without major failure and maintain life safety. Maps of the United States provide for evaluation of an effective peak acceleration and velocity. The design level return time varies with the type of building system; for a steel frame, it is about 85 years. Buildings are grouped according to importance and occupancy. Seismic performance categories are established. Site effects are considered by three soil profiles with corresponding base shear factors of from 1.0 to 1.5. Provisions for both lateral force procedures and dynamic analysis exist. Irregular structures require a dynamic analysis. Load factors are established as

$$U = 1.2\,D + 1.0\,L + 1.0\,S \pm 1.0\,E$$

$$U = 0.8\,D \pm 1.0\,E$$

Orthogonal effects are considered by combining 100% of the forces for one direction plus 50% of the forces for the orthogonal directions. Story drifts are set at 0.010 for critical facilities and 0.015 for ordinary buildings. Factors are used to increase computed drifts for possible inelastic behavior. For the lateral force method, the base shear is given by

$$V = C_s\,W$$

where $$C_s = \frac{1.2\,A_v\,S}{R\,T^{2/3}} \text{ or } \frac{2.5\,A_a}{R}$$

A_a = effective peak acceleration

A_v = effective peak velocity

S = soil profile factor

R = response modificaton; depends on type of building system

T = building period

The base shear is distributed based on the floor weights and heights. Overturning is considered. Story drifts are multiplied by factors based on the building system to compare with allowables and are limited to control significant yielding. Under the prescribed loading, deformations should be less than the level causing complete plastification of at least the most critical region of the structure. This would be the formation of the first plastic hinge in a steel frame. In a concrete frame this point is reached when the critical member reaches its ultimate strength. Redundant members are assumed, so plastification of other members is prevented and a complete failure mechanism is also prevented.

Ultimate strength concrete design or working stress steel design are allowed. In the latter case, allowable stresses are increased by 1.7 to approach ultimate strengths. For modal analysis, the spectral levels are based on the base shear equation given previously. Modes are combined by square root of the sum of the squares of individual mode shapes.

Comments on Present Methods

Conventional design practice assumes that governing ground motion occurs as horizontal translation (vertically propagating shear waves). The ground motion actually has six components - three translational and three rotational. The building may see spatial variation and motions that are not in phase. In many cases, design against collapse is the governing requirement rather than service loading. Especially in this area, use of linear elastic techniques for response and use of inelastic response spectra generated by application of ductility factors to elastic spectra may be in significant error.

Reference 15 reports on a study of a four-story building in which interstory displacements were evaluated. Table 3 gives response from elastic analysis using 39 real earthquakes, 15 artificial earthquakes, and the average response spectrum. Although the mean values agree for this elastic analysis, the coefficient of variation is large. Table 4 gives interstory ductility ratios for inelastic analysis; again, variation is large. This illustrates the difficulty in designing for a specified level of yielding. Current equivalent static forces, procedures, and elastic analyses are lacking exactness. It is important to point out that ductility ratio or yielding varies throughout the building. The use of spectra for multidegree-of-freedom systems when hysteretic behavior in the structure occurs can significantly underestimate damage. The hysteretic behavior results in strength degradation not accounted for in elastic-plastic models. The use of inelastic design response spectra determined from linear elastic spectra has serious limitations in that the duration of the ground shaking and the number and characteristics of the acceleraton pulses are omitted. Repeated large acceleration pulses can lead to accumulation of large strains. Further, the type of excitation which induces dynamic response in a linear elastic system is very different from the type of excitation which is critical to an elastoplastic system (Ref 16). Resonance phenomenon is of major significance in an elastic system; however, small inelastic deformations in a yielded system are equivalent to large values of damping. The natural period of a structure changes with deformation (Figure 2). Use of linear elastic response spectra in elastic design is controlled by the resonance phenomenon induced by single acceleration pulses with the same periodicity as the structure. Considerably larger deformations can be produced by just one long pulse with an effective acceleration exceeding the yield strength of the structure. For inelastic response, the largest incremental velocity - rather than the largest peak acceleration - is of importance (Ref 16). Thus, the type of ground motion which is critical depends on the type of behavior of the structure.

Building Damage

A correlation exists between the degree of damage and the intensity of shaking. Table 5 shows a damage probability matrix for buildings in the 1971 San Fernando earthquake (Ref 10). This gives an overview of structural performance but does not relate structure design parameters. In a similar way, Culver et al. in Reference 17, knowing the quality of construction in terms of strength, physical condition, integrity, and workmanship, estimate drift to yield and ductility to failure (Tables 6 and 7). These set up allowable deflection guides for comparison with results of an analysis.

Masonry buildings as a class have been studied to relate damage (mean damage ratio) to intensity of shaking.

Masonry buildings are classified into four groups:

- Masonry A: Engineered reinforced masonry with good materials and good workmanship; designed to resist earthquakes
- Masonry B: Reinforced masonry with good workmanship and materials; not designed to resist earthquakes
- Masonry C: Unreinforced masonry with ordinary workmanship and materials; not designed to resist earthquakes (Type III buildings)
- Masonry D: Poor materials, such as adobe, and poor workmanship; little or no earthquake resistance

Figure 3 shows the damage as a function of intensity (Ref 7). In Figure 3a an attempt is made to identify the subsystems of the structure and identify the damage to each subsystem.

Another study by Sauter (Ref 18) also gives damage ratio as a function of intensity (Figure 4). Data in Figure 3 show average behavior whereas data in Figure 4 tend to be higher such as would be used for insurance estimates.

In evaluating the total loss to a facility from an earthquake, one must include the physical damage, the injury and loss of life, the damage to contents of the building, and the interruption in the functional use of the facility and its associated cleanup. In evaluation of the physical damage, the cost of repair may exceed the original cost of the structure; thus, the damage ratio (percent of damage) must reflect the present worth or replacement of the facility.

Physical damage involves both the structural elements and the nonstructural. Nonstructural elements include interior and exterior walls, partitions, ceilings, plumbing, glazing, lighting fixtures, stairs, electrical systems, and elevators. These may represent a larger monetary loss than the damage to the structural system. Such is usually the case with steel frame buildings. Walker (Ref 19) relates story drift to total damage and nonstructural damage (Figures 5 and 6).

Other costs include damage to the contents and downtime; this may represent 30% of the total loss.

The intensity may be related to site acceleration through use of Figure 7. Figure 8 illustrates conceptually how it is possible to use previous data to determine damage as a function of site acceleration. Once the function has been established for each type of construction, it

is possible to divide the acceleration by some form of design level acceleration and thus normalize the acceleration axis. Figures 9, 10, and 11 illustrate this. Several levels of design acceleration were selected and are shown with the design ductility levels. The relationship between damage and acceleration ratio is seen to be approximately linear.

Other Losses

Loss of life is especially difficult to quantify. Figure 12 will be adopted for this study (Ref 20). The loss of life is estimated, based on historical evaluation of previous earthquakes from work by Wiggins (Ref 21).

$$\text{Numbers of lives lost} = \frac{(\text{Building \$ Loss})^{0.813}}{100{,}000}$$

The number of injuries may be estimated as follows:

$$\text{All injured} = 43.0 \times \text{loss of life}$$

$$\text{Seriously injured} = 2.8 \times \text{loss of life}$$

Cost Increase of Earthquake Resistant Construction

The basic costs of seismic resistant design are found in the structural system, specifically the beam girders and columns. Other costs include the foundation.

Nakano (Ref 22) investigated structure costs. For elastic design with seismic design coefficients and allowable stress limits, he found the following cost ratio.

$$\gamma = 1 + R\,(C - 0.2)$$

where γ = cost ratio $\left(\frac{\text{cost by seismic design coefficient} = C}{\text{cost by seismic design coefficient} = 0.2}\right)$

R = cost coefficients as given in Table 8

C = seismic design coefficient ($0.2 \leqq C \leqq 1.0$)

Two examples are shown - Figure 13a for steel frame buildings and Figure 13b for reinforced concrete buildings.

Whitman et al. (Ref 23) note the increase in cost for typical apartment buildings for various UBC design levels (Figure 14). These data appear to be much lower (about threefold) than Nakano's. Whitman's data are more appropriate to United States construction.

The Department of Housing and Urban Development (HUD) sponsored a study investigating seismic design costs of high-rise residential structures (Ref 24). They concluded that upgrading typical high-rise residential construction to seismic requirements of the UBC varied a great deal from city to city. Approximately 0% to 30% was added to the basic cost, depending upon the existing local code. The total deadweight of the building was found to be an important factor in the general cost of

construction - the lighter the building the smaller the costs of upgrading. Figure 15 presents some of their data, which are somewhat higher than Whitman's.

Leslie and Biggs (Ref 25) analyzed a 13-story steel frame building. Figure 16 shows the breakdown of costs of structural and nonstructural items - Table 9 showing the costs for nonstructural items and Figures 17 and 18 for structural items. The costs to nonstructural systems are seen to be low. The major factor in the nonstructural system is providing proper anchoring and bracing. Restraint of equipment was provided to prevent sliding. Restraints and bracing were provided for ventilating ducts, plumbing, transformers, and switchgear. Restraints were provided for elevator motors. It should be noted that the deadweight of this structure is low so the resulting costs of increasing seismic resistance will be less. Reinforced concrete structures would exhibit increased cost ratios, perhaps as much as a 75% increase in cost as illustrated in Figure 19.

EARTHQUAKE COST DAMAGE ANALYSIS

An analysis of expected damage, injury, and cost of earthquake resistant design for various design levels was made using the preceding information. Site acceleration probability distributions for San Diego, Calif.; Memphis, Tenn.; Bremerton, Wash.; Long Beach, Calif.; and Port Hueneme, Calif.; were used. Typical results are shown in Table 10 and Figure 20. Table 10 gives the design acceleration level, the probabilities that the acceleration will and will not be exceeded, the associated damage for design at that level, and the construction cost increase for designing to that level. The last column gives the total cost, including injury. Figure 20 gives a plot of design acceleration and total cost increase where the total cost includes construction increase and expected damage and injury. A minimum cost in this example occurs for a design acceleration of 0.17 g which has a return period of 357 years. Numerous analyses were performed, using the probability distributions for the above-mentioned sites. Results are shown in Figures 21 and 22. Figure 21 shows the least-cost design acceleration as a function of the 225-year return time acceleration. The 225-year* acceleration characterizes the site seismicity sufficiently accurate for all the sites, and the data give a clear trend with minimal scatter which is significant considering the variation in locations.

Figure 21 shows that for a 225-year acceleration of 0.2 g the least-cost design acceleration would be about 0.19 g. The term "design acceleration" in this case implies a design of a steel building to about yield level (see Figure 10). The selection of the design level is based on the ratio of collapse to design level and its associated ductility and damage as illustrated in Figure 10.

Figures 23 and 24 show plots similar to the preceding one with the design level varied: elastic design, ductility of about 1.5, and ductility of about 2.5. These curves are not intended to be generalized for use in design but rather to illustrate the concept that the type of design or amount of inelasticity to which a structure is designed influences the least-cost design. The specification of a 130-year elastic

*Twenty percent chance of exceedence in 50 years.

design acceleraion or a 160-year inelastic ductility equal to 1.5 design acceleration should produce the same definition of structure, provided consistent procedures are employed. This is admittedly an idealization since two actual earthquakes, even at the same nominal acceleration, will not produce the same response as shown in the preceding section on comments of present methods. The thought is that the same performance by alternative levels of inelastic design may be specified.

DISCUSSION OF STUDY RESULTS

First, it is important to point out that the data used in this study are an accumulation of average behavior and have a wide scatter. The intent is to demonstrate a trend, not to give guidance for a specific building.

An examination of Figure 21 shows that the data are curved away from the 1:1 correspondence line. The optimal design is not related to a unique return period. Rather, at low accelerations it is greater than the 225-year return time, and at high accelerations it is less. This agrees with basic structural engineering experience in that it is difficult and costly to resist high level shaking. Though uncertainty is present in the data, it is believed the uncertainty will basically shift the curve to left or right but not change its slope. So, the basic premise that a constant return time is not a general optimal design level should remain.

The basic data indicate that optimal design in high acceleration areas should be greater than that prescribed by building codes but less than the 225-year return time, which is the current NAVFAC criterion.

DISCUSSION OF RISK

If proposed government regulations are placed in effect, two important criteria for establishing priorities of methods to reduce risk are (1) the perception of the probability of the catastrophic event and (2) the net cost of the saved lives and reduced damage. From an economic viewpoint, an acceptable decision exists whenever the benefits minus the costs are positive. The optimal decision level maximizes the net present value (Ref 25).

An example can be seen in the following. A building may be strengthened at a cost of $10,000, reducing seismic damage of $2 million. The probability of damaging earthquakes occurring is 0.002. Should the building be strengthened? The expected benefit is $4,000 (0.002 x $2 million). The expected benefit is less than the cost; thus, the economic decision is no.

In general, the purchase of insurance violates economic theory since the insurance policy premium exceeds the expected benefit; however, substantial insurance is sold. Risk aversion is accomplished by the payment of a fixed cost to avoid the potential of large losses. The perception of the loss may be linearly or nonlinearly related to the cost, based on the perception of the investigator (Ref 25). For example, if the $10,000 cost of strengthening the example building is viewed as an acceptable business expense and if the possibility of the total loss

of the facility is an unacceptable catastrophe, a businessperson may indeed opt for the strengthening option. It is realized that it is not economically advantageous but the consequence is perceived in a "nonlinear" manner.

A large organization such as the Navy is regionally dispersed, therefore, the relationship of dollar loss from an earthquake to cost of strengthening for such an organization should be linear. When the case of 100 buildings similar to the example building being widely dispersed is considered, it might be assumed that the full loss of these 100 separated buildings is unlikely (100 buildings x $2 million). Thus, the expected value is indeed a good representation of the actual loss. This illustration demonstrates risk pooling, which is applicable to large organizations like the Navy.

NAVY ECONOMIC ANALYSIS

Reference 27 specifies procedures for economic analyses of facilities. The principles of the analyses are:

1. To insure an optimum allocation of scarce resources
2. To effectively consider alternatives and life-cycle funding implications
3. To recognize that money has value over time expressed by an interest rate

This problem thus must include the consideration that earthquake strengthening is expressed as a current cost increase to protect against a future dollar loss. The real world is complicated by cost increases over time (referred to as inflation). This means that to repair or replace the damaged building some time in the future will cost more than today. The work in the previous sections expressed costs of strengthening and damage as a percentage of building value to maintain a common reference. That premise recognized increased value of the building and increased costs of repair. In an economic sense this may be expressed as letting the discount rate (the value of return on investment) be equal to the inflation rate.

The government has placed a value on money in time. Reference 27 and DODINST 7041.3 specify the discount rate as 10%. Reference 27 states:

> "The rationale for adopting the private-sector rate of return as the discount rate for analyzing Government investment proposals turns on the notion that Government investments are funded with money taken from the private sector (preponderantly via taxation), are made in the ultimate behalf of the private sector (i.e., the individuals comprising it), and thus bear an implicit rate of return comparable to that of projects undertaken in the private sector. In this interpretation, 10% measures the opportunity cost of investment capital forgone by the private sector."

The 10% rate is a differential rate in addition to inflation (further, the maximum economic life is set at 25 years for permanent buildings; the present earthquake design criteria are based on 50 years).

When the present worth of the annual expected damage is considered using a discount rate of 10%, the present worth estimate of the damage would effectively be reduced by a factor of about 5. To restate this, the earthquake could occur at any point during the life of the structure; the best estimate is to consider an annual series of expected losses. The present worth of this series can be computed and its value is about one-fifth of the total expected loss. This has a major effect on the optimum design levels (Figures 25 and 26).

It is important to note that the discount rate specified for use is actually a differential rate of 10% over the rate of inflation. It is recognized that the future cost of the repair would increase with time. One could use the differential rate and not consider inflation, or one could consider the rate of inflation to project an increased repair cost and then discount that cost using a discount rate of 10% plus the inflation rate. The results for modest inflation rates are approximately the same. The differential cost approach has been used in this study.

Comparing the NAVFAC acceleration criterion (80% probability of nonexceedance in 50 years) with the optimal least cost-acceleration it is possible to compute cost per life saved (Figure 27). For example, for a 50-year exposure of a building with an initial cost of $1 million:

A. <u>NAVFAC Criterion</u>

Acceleration	0.26 g
Cost of seismic strengthening	$338,700
Present worth of expected damage	$26,200
Lives lost	0.0491 people

B. <u>Least Cost Design</u>

Acceleration	0.12 g
Cost of seismic strengthening	$123,900
Present worth of expected damage	$95,100
Lives lost	0.1711 people

(1) Damage Difference (B - A)	$68,900
(2) Cost Difference (A - B)	$214,800
(3) Lives Lost (B - A)	0.122
Marginal Value [(2) - (1)]/(3)	$1,200,000

A marginal value – $1,200,000 – for the current NAVFAC earthquake criterion of 80% probability of not being exceeded in 50 years is high in comparison with other sectors.

Paté (Ref 28) has calculated the marginal value per life which is necessary if benefits are to exceed costs for seismic design level in the San Francisco Bay area. Results, although tentative, indicate that

to adopt the 1973 UBC a value of life would be on the order of $6 million/life for the enforcement of the seismic provisions in new buildings and $22 million/life for the upgrading of old buildings to the same standards. A value of $20 million/life is required to adopt the 1976 provisions for new construction. The marginal cost per life saved in other sectors of public safety is much lower. In the transportation sector, cost benefit analysis projections are around $300,000. The conclusion of Paté's work is that it is hard to justify the 1973 and 1976 seismic design provisions solely on the interest of life safety.

Organizational theory suggests that an individual within an organization accepts a framework which defines the areas of responsibilities and correspondingly narrows the scope of alternative decisions. Engineers in seismic design are not usually tasked to resolve problems from the viewpoint of investment costs and protection benefits. Engineering regulations from as far back as the Code of Hammurabi have had public safety foremost. The public perceives a requirement for the government to assure safety. Yet, according to Paté, the value of life in other sectors such as health is markedly lower than imposed for building safety; take, for example, the $20,000/life value as a result of the program to reduce the risks of heart attack. The perceived action results from the public and the professional engineer's risk aversion from the visibility of large catastrophes. Public policy may also have a cyclic aspect as people forget about earthquakes, where return periods are larger than collective memory, and perception diminishes.

GROUP RISKS

A major point in an analysis of this type is: given the earthquake, can the Navy accept the loss of an installation? Table 11 shows a damage summary for key buildings at the North Island Air Station, San Diego, taken from Reference 29. Figure 28 illustrates the probability of not exceeding a given level of dollar loss based on Table 11, while Figure 29 is derived from Reference 2. A 50-50 chance exists in 50 years of a loss of about $30 million which has a present worth of $6 million (Figure 28). It would cost about $13 million to design against this 0.2 g level of ground motion.

SUMMARY

Current NAVFAC criteria specify that for important buildings a site seismicity study shall be performed and the design earthquake level shall be taken as the site acceleration having an 80% chance of not being exceeded in 50 years (225-year return time). This report has reviewed construction cost increases for seismic strengthening and expected damage from seismic shaking. The specification of a 225-year return time acceleration does not produce optimal least total cost designs over all ranges of acceleration. Rather the least total cost design acceleration varies with site activity. It is not economically advantageous to design against high ground acceleration. Use of economic analysis procedures specified in Reference 27 suggest the present worth of future damage is low enough that an earthquake design return time of

half the present value for a ductility of 1.0 is more efficient (see Figures 25 and 26). Collapse event* return times of about 1,000 years seem appropriate and consistent with the design levels, based on an individual exposure of 1×10^{-5} fatalities per year.

It is important to note that should reductions in design (or service) levels be made, it is essential that collapse level investigations be performed to insure adequate ductility.

In evaluating seismic planning decisions, it is recommended that the Navy should:

1. Adopt a risk pooling policy, relating a direct linear relation between expected losses and cost of strengthening

2. Consider as acceptable investments only instances where the benefits exceed the costs

Again, it is important to note that by reducing design levels, immediate cost savings will occur; however, increased damage in the future is expected. The present worth of this future damage, using the DOD-specified differential rate of return, is less than the costs of strengthening. However, in conjunction with this must be the realization that more extensive disruption will occur. Present NAVFAC criterion exceeds building codes by two to three times; whereas optimal design, as suggested herein, would be approximately at the same levels.

A basic analysis of typical Navy buildings (such as Bachelor Officer Quarters and industrial and administrative buildings) should be made. Typical structures should be designed for various acceleration levels and cost estimates made to evaluate cost at various seismic levels as a function of design acceleration. The structures should also be analyzed to better define the specific damage function for that type of construction to give a better estimate for a specific case study than that given in the general data used herein.

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Table 1. California Earthquake Insurance Rates, Building Rates per $100 Coverage (Insurance Services Office)(from Ref 8)

Type of Construction	Class of Risk	Mandatory Deductible	Insurance Rates in Zones --		
			1	2	3
Small wood frame structures as dwellings not over 3,000 ft^2 and not over three stories	I	5%	0.11	0.15	0.23
One story, all steel. Single or multistory steel frame, concrete fireproofed, concrete exterior panel walls, concrete floors and roof--moderate wall openings (otherwise Class V).	II	5%	0.19	0.25	0.38
Single or multistory concrete frame, concrete walls, floors and roof--moderate wall openings (otherwise Class VI).	III	5%	0.23	0.30	0.45
Large area wood frames and other wood frames not falling in Class I.	IV	5%	0.25	0.35	0.53
Single or multistory steel frame, unreinforced masonry exterior panel walls, concrete floors, and roof.	V	5%	0.25	0.35	0.53
Single or multistory steel frame, unreinforced masonry exterior panel walls, concrete floors, and roof.	VI	5%	0.30	0.40	0.60
Walls of cast in place or precast reinforced concrete reinforced brick, reinforced concrete block, or reinforced brick, with floors or roof other than reinforced concrete. Reinforcing must be adequate.	VII	10%	0.56	0.75	1.12

(continued)

Table 1. Continued

Type of Construction	Class of Risk	Mandatory Deductible	Insurance Rates in Zones --		
			1	2	3
Bearing walls or unreinforced adobe, hollow clay tile, or unreinforced hollow concrete block.	VIII	15%	$1.87	$2.50	$3.75
Buildings which can resist earthquake of 1906 type with minimum to slight property damage.	Special Rate	5%	*	*	*

NOTES: All rates quoted in this table require 70% coinsurance. Rates in this table are for the Earthquake Damage Assumption Endorsement. All *buildings during the course of construction* in California are placed in one of the following classifications: I, IV, V, VI, VII, or VIII. Rates given in this table are for use with the mandatory percentage deductible. To obtain rates for other optional percentage deductible, reduce rates shown in table for each percent of deductible in excess of the mandatory percentage as follows: 2% on Class I to VI and Class-Special Rate, and 1% on Class VII and VIII. The maximum percentage deductible permitted is 40%.

Table 2. Ductility and Damping Levels (Ref 12)

Type of Structure	Damage		Collapse	
	Ductility	Damping (%)	Ductility	Damping (%)
Space Frame Ductile Moment Resisting-Concrete (Code K = 0.67)[a]	1.5	10	4.0	10
Space Frame Ductile Moment Resisting-Steel (Code K = 0.67)	1.5	10	4.0	10
Ductile Moment Resisting Concrete Frame with Concrete Shear Walls (Code K = 0.80)	1.5	10	4.0	10
Ductile Moment Resisting Steel Frame with Vertical Bracing (Code K = 0.80)	1.5	7	2.0	7
Concrete Shear Walls with Vertical Load-Bearing Frame (Code K = 1.00)	1.5	7	2.0	7
Vertical Load Steel Frame With Vertical Bracing (Code K = 1.00)	1.5	5	2.0	5
Flexible Diaphragms on Masonry or Tilt-up Walls (Code K = 1.33)	1.5	5	2.0	5

[a]K is from UBC.

Table 3. Peak Inter-Story Elastic Displacements of Four-Story Building (Ref 15)

[Fundamental Period = 1.13 seconds]

Story	Time-History Analysis of 39 Earthquakes[a]	SRSS[b] Modal Analysis – Mean (or Mean + σ) Response Spectrum	Time-History Analysis of 15 Artificial Motions[c]
		Mean	
1	0.122	0.126	0.133
2	0.107	0.104	0.115
3	0.092	0.088	0.093
4	0.063	0.059	0.064
		Mean + σ	
1	0.194	0.193	--
2	0.169	0.166	--
3	0.137	0.131	--
4	0.089	0.083	--
		Coefficient of Variation	
1	0.58	--	0.25
2	0.57	--	0.29
3	0.48	--	0.29
4	0.40	--	0.24

[a]Normalized to 0.3-g peak ground accelerations.

[b]Square root of sum of squares.

[c]All generated from mean response spectrum.

Table 4. Peak Inter-Story Ductility Ratios of Four-Story Building (Ref 15)

[Fundamental Period = 1.13 seconds]

Story	Analysis of 39 Earthquakes[a]	Analysis of 15 Artificial Motions[b]
Mean		
1	5.7	4.4
2	2.6	3.2
3	4.0	5.0
4	9.7	13.8
Coefficient of Variation		
1	1.23	0.42
2	0.48	0.31
3	0.48	0.28
4	0.49	0.39
Maximum - Minimum		
1	38.6 - 0.8	10.2 - 2.6
2	5.9 - 0.8	5.6 - 1.9
3	8.9 - 1.0	7.6 - 2.9
4	27.8 - 2.2	21.1 - 7.0

[a]Normalized to 0.3-g peak ground acceleration.

[b]All generated from mean response spectrum.

Table 5. Damage Probability Matrix for Post-1947 Buildings, From San Fernando Earthquake (based on Ref 10)

Damage State[a]	Intensity		
	VI	VII	VII-VIII
0	79%	33%	6%
1	18%	34%	19%
2	3%	20%	44%
3	-	10%	13%
4	-	3%	6%
5	-	-	12%
6	-	-	-
7	-	-	-
8	-	-	-
Mean Damage Ratio	0.05%	0.5%	2.74%
Number of Buildings	57	156	16

[a]0 No Damage

1 Minor nonstructural damage--a few walls and partitions cracked, incidental mechanical and electrical damage

2 Localized nonstructural damage--more extensive cracking (but still not widespread); possibly damage to elevators and/or other mechanical electrical components

3 Widespread nonstructural damage--possibly a few beams and columns cracked, although not noticeable

4 Minor structural damage--obvious cracking or yielding in a few structural members; substantial nonstructural damage with widespread cracking

5 Substantial structural damage requiring repair or replacement of some structural members; associated extensive nonstructural damage

6 Major structural damage requiring repair or replacement of many structural members; associated nonstructural damage requiring repairs to major portion of interior; building vacated during repairs

7 Building condemned

8 Collapse

Table 6. Quality Rating of Materials in Structural System (Ref 17)

Material	Quality		
	Good	Average	Poor
a. Strength			
Structural Steel and Metal Decking	$f_y \geqq 40$ ksi Double sheet metal decking	$f_y \geqq 30$ ksi Single sheet metal decking	$f_y < 30$ ksi Cast iron Corrugated iron
Concrete: Including Precast and Prestressed	$f'_c \geqq 3$ ksi	$f'_c \geqq 2$ ksi	$f'_c < 2$ ksi
Masonry (based on core tests or specified strengths)	$f'_m \geqq 2.0$ ksi Mortar $f'_c \geqq 2.0$ ksi Grout $f'_c \geqq 2.0$ ksi Continuous inspection	$f'_m \geqq 1.2$ ksi Mortar $f'_c \geqq 1.0$ ksi Grout $f'_c \geqq 1.0$ ksi Called inspection	$f'_m < 1.2$ ksi Mortar $f'_c < 1.0$ ksi Old sand-lime mortar Grout $f'_c < 1.0$ ksi No inspection
Timber Plywood	$f_b \geqq 1.9$ ksi Select structural Structural I plywood	$f_b \geqq 1.5$ ksi Construction Industrial Structural II plywood	$f_b < 1.5$ ksi Not grade marked Plywood not grade marked
Gypsum	$f'_g \geqq 1.0$ ksi	$f'_g \geqq 0.5$ ksi	$f'_g < 0.5$ ksi

(continued)

Table 6. Continued

Material	Quality		
	Good	Average	Poor
b. Physical Condition			
Structural Steel and Metal Decking	No weld cracks No cracks at holes No corrosion	Few cracked welds (none critical) Few cracks at holes (none critical) Slight corrosion Machine bolts	Many cracked welds Many cracks at holes Moderate corrosion
Concrete: Including Precast and Prestressed	Few minor shrinkage cracks No shear of flexure cracks No excessive deflection (i.e., drift $\leq$ story height divided by 240)	Few shear or flexure cracks (none critical) Few shrinkage cracks Few cracked welds at precast connections	Many shrinkage cracks Many shear and flexure cracks; deteriorated concrete; exposed reinforcing; excessive deflections in beams and slabs; many cracked welds
Masonry	Few minor shrinkage cracks No shear or flexure cracks Plumb walls	Few moderate shrinkage cracks Few shear or flexure cracks	Many shrinkage cracks Many shear and flexure cracks Deteriorated, soft mortar Exposed reinforcing; bowed and out-of-plumb walls
Timber and Plywood	No splits or twisted members No loose bolts or screws No loose knots No projecting nails on bottom side Grade worked lumber	Few knots and splits, twisted members Few loose bolts and screws Minor-moderate deflections Few loose nails Fair nailing pattern Fair connections	Many splits and twisted members Many loose bolts and screws Rotting; excessive deflections; many loose nails

(continued)

Table 6. Continued

Material	Quality		
	Good	Average	Poor
b. Physical Condition continued			
Gypsum	No cracks in formboard Good connection details Smooth hand surface	Few cracks in formboard Crack pattern over T-supports	Many cracks and excessive deflection of formboard
c. Integrity			
Structural Steel	All parts of joints in full contact; members straight Structure plumb; bolts tight; structural welds all OK; deck welds all OK Continuous inspection	Few joints with members not in full contact; few bent members; few loose bolts Few poor welds Selective inspection	Many joints with members not in full contact; many bent members; many loose bolts Many poor welds; no inspection
Concrete Including Precast and Prestressed	Clean constructon joints No rock pockets Construction straight and plumb. All tendons grouted Continuous inspection	Few poor construction joints; few small rock pockets; few members show evidence of form failure; few poor welds at precast joints Tendons not grouted Called inspection	Many poor construction joints; many rock pockets Many evidences of form failure Mixture of hardrock and lightweight concrete at joints; many poor welds No inspection
Masonry	All grout and mortar spaced filled Construction straight and plumb Running bond Continuous inspection	Few grout and mortar spaces not filled Running bond Called inspection	Many grout and mortar spaces not filled Construction bowed and out of plumb Stacked bond No inspection

(continued)

Table 6. Continued

Material	Quality: Good	Quality: Average	Quality: Poor
d. Workmanship continued			
Concrete: a. Poured in place b. Precast c. Prestressed	Close spacing of ties and stirrups Ductile reinforced details Spiral type columns; no precast or prestressed members	Ordinary reinforcing details; tied columns (no. 3 and over) Poured precast joinery well detailed with welded reinforced precast and prestressed members	Deficient framing details No mild steel Deficient reinforcing details; tied columns (no. 2 and smaller) Welded precast joinery
Masonry	Fully grouted members; embedded anchors, bolts, and strap ties; adequate reinforced, uniformly spaced in two directions; all columns filled; adequate laps at corners and intersections; adequate bars at openings	Bolted connections; adequately reinforced, concentrated at top and bottoms of walls; columns filled at reinforcing only; horizontal mesh reinforced	Non-grouted wall, nailed connection; no or partial reinforcement; poor tie and lap details; filler walls not anchored to framing
Timber	Strap anchors to masonry walls plus shear transfer connections	Melar hardware at some connections; few steel strap ties at joints	No connections to masonry walls; no strap at connections
Plywood	Bolts at critical joints well anchored to footings Steel strap ties to walls spaced 4 ft or less	Bolted and nailed joints anchored to footings No strap ties or straps over 4 ft on-center	No anchored to footings No strap ties
Gypsum	Trussed purlins; adequate connection to walls; mesh reinforced	Solid purlins; poor connections to walls	No reinforcement; no connections to walls

(continued)

Table 6. Continued

Material	Quality		
	Good	Average	Poor
e. Nonstructural Components			
Ceilings	Gypsum board and metal lathe attached directly to structural framing (not suspended)	Suspended wood framing with Gypsum board nailed Suspended metal lathe and plaster Suspended plywood nailed to wood framing	Suspended T-bar with lay-in or splined acoustical tiles
Partitions	Wood panel, well-anchored and braced to structure	Gypsum board and metal studs and plaster anchored and braced to structure	Unreinforced masonry and gypsum block Ceiling height partitions, braced by suspended T-bar ceilings
Trim and Veneer	Not applicable	Masonry veneer and facings, well-anchored and with cement mortar	Masonry veneer on facings, not anchored and with poor mortar Heavy ornamentation such as statues, steeples, and cornices
Glass	Full elastomeric mounting with at least 1/2 in. clearance all around. Glass set outside of framing.	Elastomeric mounting, 1/4 in. clearance, set between framing	Fixed sash with putty - negligible clearance

(continued)

Table 6. Continued

Material	Quality: Good	Quality: Average	Quality: Poor
e. Nonstructural Components continued			
Filler Walls Between Framing Members	Reinforced concrete or masonry well-anchored to frame; metal and wood studs and plaster anchored to structure	Unreinforced masonry cement mortar anchored to structural framing	Unreinforced masonry, poor mortar; no anchorage to structural framing
Curtain Walls Set Outside of Framing Line	Reinforced concrete and masonry well-anchored to structure; metal frame and siding well-anchored; struss well-anchored	Unreinforced masonry with good cement mortar Anchored to structure Precast concrete units - well-anchored to structure	Unreinforced masonry, poor mortar, not anchored to structure Precast concrete units, welded anchorage
Fire Escapes	Not applicable	Metal framing attached to building	Free standing concrete and masonry
Overhangs and Gargoyles	Not applicable	Reinforced and anchored	Unreinforced masonry, poorly anchored
Signs and Marquees	Not applicable	Steel frames Signs on roof and wall	Heavy marquees
Antennae	Steel towers guyed	Steel towers on roof not guyed	Not applicable

Table 7. Mean Values of Drift to Yield (in./in.) and Ductility for Various Quality Ratings (Ref 17)

Function	Material	Mean Values					
		Good		Average		Poor	
		Drift to Yield	Ductility to Failure	Drift to Yield	Ductility to Failure	Drift to Yield	Ductility to Failure
Frame Member	Structural Steel	0.013	18	0.008	10	0.004	4
	Concrete: Poured in Place	0.009	10	0.006	6	0.003	3
	Precast or Prestressed	0.006	8	0.003	5	0.001	3
	Timber	0.015	11	0.009	6	0.005	3
Shearwall	Concrete: Poured in Place	0.006	7	0.003	4	0.002	2
	Precast or Prestressed	0.003	4	0.002	2	0.001	2
	Masonry	0.007	7	0.005	5	0.002	2
Diaphragm	Metal Decking	0.009	11	0.005	7	0.003	3
	Plywood	0.009	10	0.006	6	0.003	3
	Gypsum	0.004	6	0.002	4	0.001	2

Table 8. Cost Coefficients (Ref 22)

Number of Stories	Steel Frame Buildings					Reinforced Concrete Buildings				
	Number of Spans		Coefficient Ratio for Following Cost Ratios--			Number of Spans		Coefficient Ratio for Following Cost Ratios--		
	x	y	1.0	0.6	0.5	x	y	1.0	0.4	0.3
5	5	3	2.0	1.2	1.0	5	3	3.2	1.25	0.95
	5	5	2.0	1.2	1.0	5	5	3.0	1.22	0.92
	∞	5	2.0	1.2	1.0	∞	5	3.1	1.23	0.93
	∞	∞	2.0	1.2	1.0	∞	∞	3.1	1.24	0.93
						∞[b]	∞	3.7	1.43	1.10
10	5	3	2.0	1.2	1.0	5	3	4.0	1.60	1.17
	5	5	2.0	1.2	1.0	5	5	3.8	1.53	1.13
	∞	5	2.0	1.2	1.0	∞	5	3.8	1.55	1.15
	∞	∞	2.0	1.2	1.0	∞	∞	4.0	1.60	1.17

[a]Cost ratio of columns and girders to total structural cost (coefficient ratio = 1.0 means that cost of columns and girders is total structural cost).

[b]Aseismic shearwall, column, and girder system.

Table 9. Cost Increase for Increasing Seismic Force Design Levels (Ref 25)

Item	Percentage Total Cost of Item (%)	Increase Over Original Total Construction Cost (%) for Zones --			
		1	2	3	4
Structural					
Structural Steel[a]	12.5	-	0.171	2.68	5.4[b] 6.6[c]
Foundation[a]	1.2	-	0.162	0.162	0.162
Concrete Walls and Slab[a]	3.7	-	0.024	0.09	0.102
Composite Deck[a]	5.8	-	-	-	-
Total Structural	23.2	-	0.36	2.93	5.66[b] 6.86[c]
Nonstructural					
Masonry Core[a]	5.0	0.55	0.55	0.55	0.71
Precast Panels[a]	4.73	-	0.024	0.033	0.067
Plumbing	3.6	-	-	0.022	0.022
HVAC	18.8	-	0.032	0.15	0.197
Electrical (including lights)	8.5	-	0.022	0.104	0.104
Elevators	4.8	-	0.027	0.131	0.165
Window Systems	4.5	-	-	-	0.627[d]
Partitions	4.45	-	0.012	0.163	0.163
Acoustical Ceilings	2.2	-	-	0.114	0.163
Miscellaneous Metals	3.7	-	0.07	0.089	0.089
Total Nonstructural	68.35	0.55	0.74	1.36	2.31
Total Code Design (code design items)	32.93	0.55	0.934	3.513	6.437[b] 7.637[c]
Maximum Design[e]	91.55	0.55	1.10	4.29	7.97[b] 9.17[c]

[a]Code design.
[b]Using builtup members.
[c]Using WF 27 girders.
[d]New window system.
[e]Total structural and nonstructural items.

Table 10. Analysis Results

[100 % damage ratio 4.00
Building value 1000000.00
Exposure period 50.00]

ACC	P	PNOT	DAM	COST	TC
0.4320	0.0000	1.0000	0.0000	0.6930	0.6930
0.4278	0.0100	0.9900	0.0001	0.6841	0.6842
0.3929	0.0198	0.9802	0.0016	0.6099	0.6114
0.3197	0.0296	0.9704	0.0071	0.4543	0.4614
0.2663	0.0392	0.9608	0.0142	0.3408	0.3550
0.2651	0.0488	0.9512	0.0144	0.3383	0.3527
0.2455	0.0582	0.9418	0.0193	0.2967	0.3160
0.2449	0.0676	0.9324	0.0195	0.2955	0.3149
0.2266	0.0769	0.9231	0.0261	0.2564	0.2825
0.2167	0.0861	0.9139	0.0304	0.2355	0.2659
0.2011	0.0952	0.9048	0.0384	0.2024	0.2408
0.1933	0.1042	0.8958	0.0432	0.1933	0.2366
0.1930	0.1131	0.8869	0.0435	0.1930	0.2365
0.1867	0.1219	0.8781	0.0482	0.1867	0.2350
0.1706	0.1306	0.8694	0.0622	0.1706	0.2329[a]
0.1700	0.1393	0.8607	0.0628	0.1700	0.2329[a]
0.1679	0.1479	0.8521	0.0652	0.1679	0.2331
0.1626	0.1563	0.8437	0.0713	0.1626	0.2339
0.1616	0.1647	0.8353	0.0725	0.1616	0.2341
0.1566	0.1730	0.8270	0.0792	0.1566	0.2358
0.1562	0.1813	0.8187	0.0798	0.1562	0.2360
0.1518	0.1894	0.8106	0.0865	0.1518	0.2383
0.1459	0.1975	0.8025	0.0962	0.1459	0.2421
0.1279	0.2055	0.7945	0.1309	0.1279	0.2588
0.1246	0.2134	0.7866	0.1385	0.1246	0.2631
0.1220	0.2212	0.7788	0.1446	0.1220	0.2667
0.1174	0.2289	0.7711	0.1569	0.1174	0.2743
0.1156	0.2366	0.7634	0.1620	0.1156	0.2775
0.1154	0.2442	0.7558	0.1623	0.1154	0.2778
0.1076	0.2517	0.7483	0.1871	0.1076	0.2947
0.1076	0.2592	0.7408	0.1871	0.1076	0.2947
0.1045	0.2666	0.7334	0.1976	0.1045	0.3021
0.1012	0.2739	0.7261	0.2093	0.1012	0.3104
0.1001	0.2811	0.7189	0.2131	0.1001	0.3132
0.0954	0.2882	0.7118	0.2308	0.0954	0.3262
0.0944	0.2953	0.7047	0.2347	0.0944	0.3292
0.0882	0.3023	0.6977	0.2619	0.0882	0.3501
0.0876	0.3093	0.6907	0.2646	0.0876	0.3522
0.0870	0.3161	0.6839	0.2674	0.0870	0.3544
0.0868	0.3229	0.6771	0.2685	0.0868	0.3553
0.0858	0.3297	0.6703	0.2733	0.0858	0.3591
0.0854	0.3363	0.6637	0.2751	0.0854	0.3605
0.0854	0.3430	0.6570	0.2751	0.0854	0.3605

[a]Minimum.

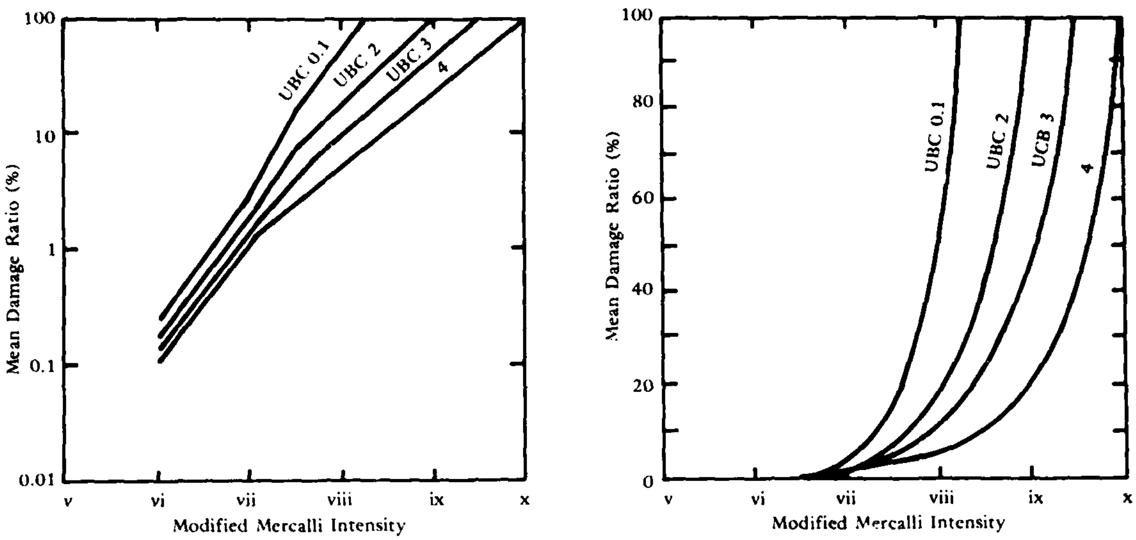
Table 11. Summary of Damage Cost Estimates for Various Levels of Maximum Ground Accelerations in g's at North Island, Naval Air Station, San Diego

Building No.	Cost of Damage ($1,000) at Following g's --										Replacement Cost ($1,000)
	0.05	0.10	0.15	0.20	0.25	0.30	0.35	0.40	0.45	0.50	
2	0	0	0	0	0	0	0	0	0	309	3,548
8A	0	0	0	0	0	248	627	997	1,345	1,465	2,000
8B	0	0	0	0	45	157	262	363	375	375	376
14A	2	23	95	133	142	151	152	152	152	152	153
14B	40	101	101	116	137	152	152	152	152	152	153
14C	40	101	101	118	139	152	152	152	152	152	153
14D	0	0	0	0	50	119	185	201	201	201	302
14E	0	0	0	0	0	0	0	0	0	0	153
14F	0	0	0	0	0	0	0	0	0	0	153
14G	0	0	0	0	0	0	0	0	0	2	153
14H	0	0	0	0	0	0	0	0	0	0	153
14I	0	0	0	0	0	0	0	0	0	0	153
33	0	0	0	0	0	0	0	0	0	0	236
68	0	0	373	950	1,330	1,600	1,737	1,758	1,758	1,758	1,759
93	0	0	0	0	0	0	0	0	0	0	3,950
306	0	0	0	165	472	781	817	845	867	869	870
318A	0	0	0	15	313	602	768	768	768	768	1,153
318B	0	0	0	0	5	318	632	945	999	999	1,000
334	0	2,711	4,491	4,491	4,491	4,491	4,491	4,491	4,491	4,491	4,492
335	0	0	1,163	1,163	1,609	1,706	1,706	1,706	1,706	1,706	1,707
472	2,407	11,718	15,739	15,739	15,739	15,739	15,739	15,739	15,739	15,739	15,740
473	0	0	0	0	0	0	0	0	0	0	169
474	0	0	0	0	28	83	106	106	106	106	107
516A	0	0	0	0	0	0	0	0	58	125	600
516B	0	0	0	0	0	0	0	0	0	0	286
651	1,962	1,962	1,962	1,962	1,962	1,962	1,962	1,962	1,962	1,962	1,963
653A	0	0	0	0	0	0	0	0	0	0	2,330
653B	0	0	0	0	0	0	0	0	0	0	100
678A	0	0	0	0	0	0	47	223	400	578	1,051
678B	0	0	0	0	138	300	456	466	466	466	700
678C	0	0	0	265	550	599	599	599	599	599	900

(continued)

Table 11. Continued

Building No.	Cost of Damage ($1,000) at Following g's --										Replacement Cost ($1,000)
	0.05	0.10	0.15	0.20	0.25	0.30	0.35	0.40	0.45	0.50	
739	0	0	0	0	0	0	0	0	0	65	764
745	0	0	0	0	72	140	186	186	186	186	280
783	0	0	269	2,500	4,179	4,444	4,444	4,444	4,444	4,444	4,445
791	0	0	0	0	0	0	0	0	0	0	300
792	0	0	0	0	0	0	0	0	0	0	610
793A	0	0	0	0	0	0	0	0	66	156	700
793B	0	0	0	10	108	202	209	209	209	209	210
Total Cost	4,451	16,616	23,481	27,627	31,509	33,946	35,429	36,464	37,207	38,034	53,872
Percent	8.3	30.8	43.6	51.3	58.5	63.0	65.8	67.7	69.1	70.6	



(a) Logrithmic Scale

(b) Arithmetic Scale

Figure 1. Damage estimate for different design strategies (after Ref. 10).

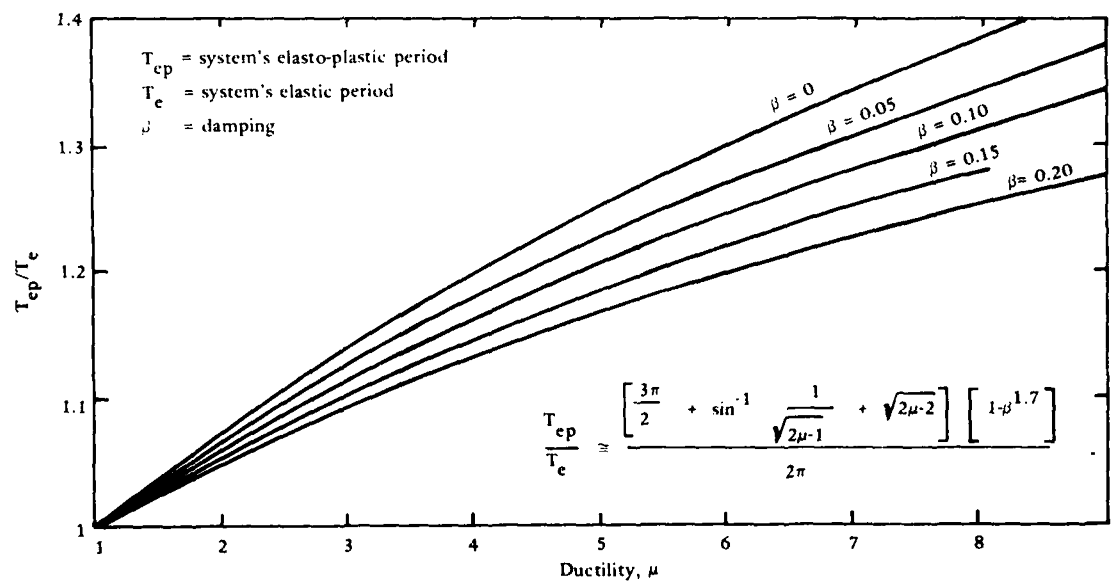
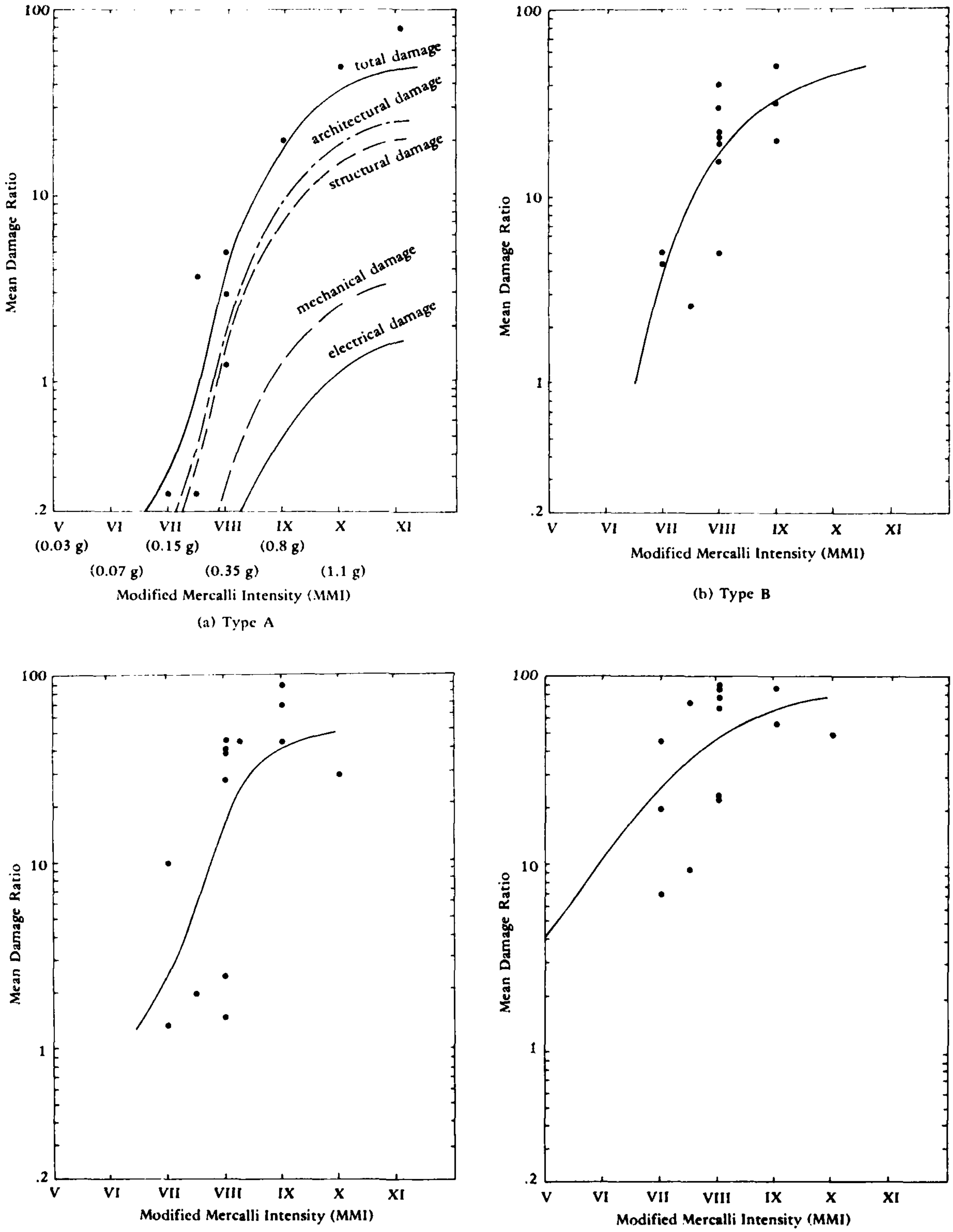


Figure 2. Ratio of elasto-plastic to elastic periods (Ref. 17).



(a) Type A

(b) Type B

(c) Type C.

(d) Type D.

Figure 3. Masonry buildings (Ref. 7 © permission to use granted by Earthquake Engineering Systems.)

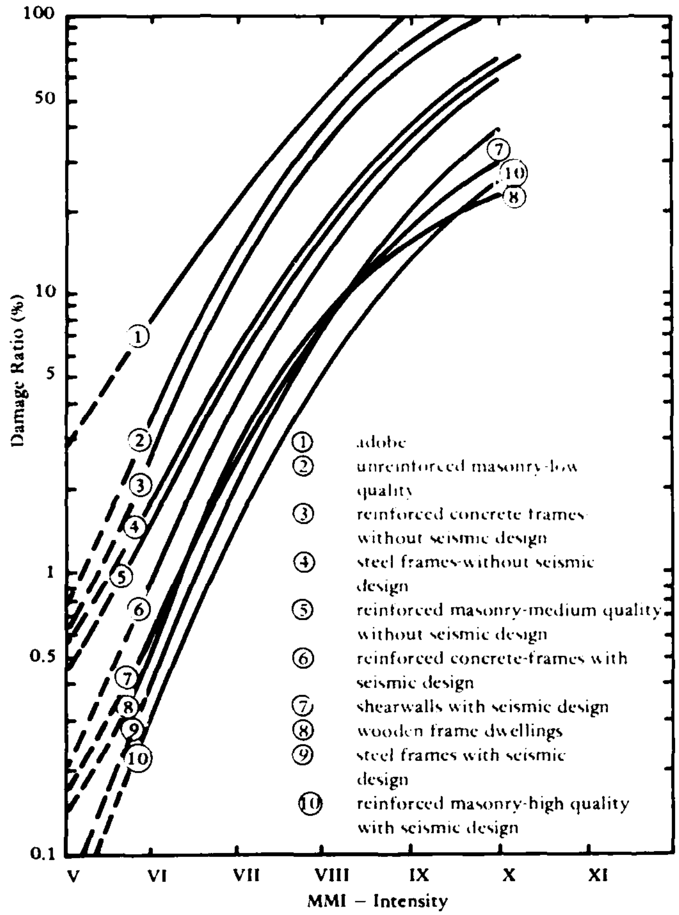


Figure 4. Average damage ratio relationship (Ref 18).

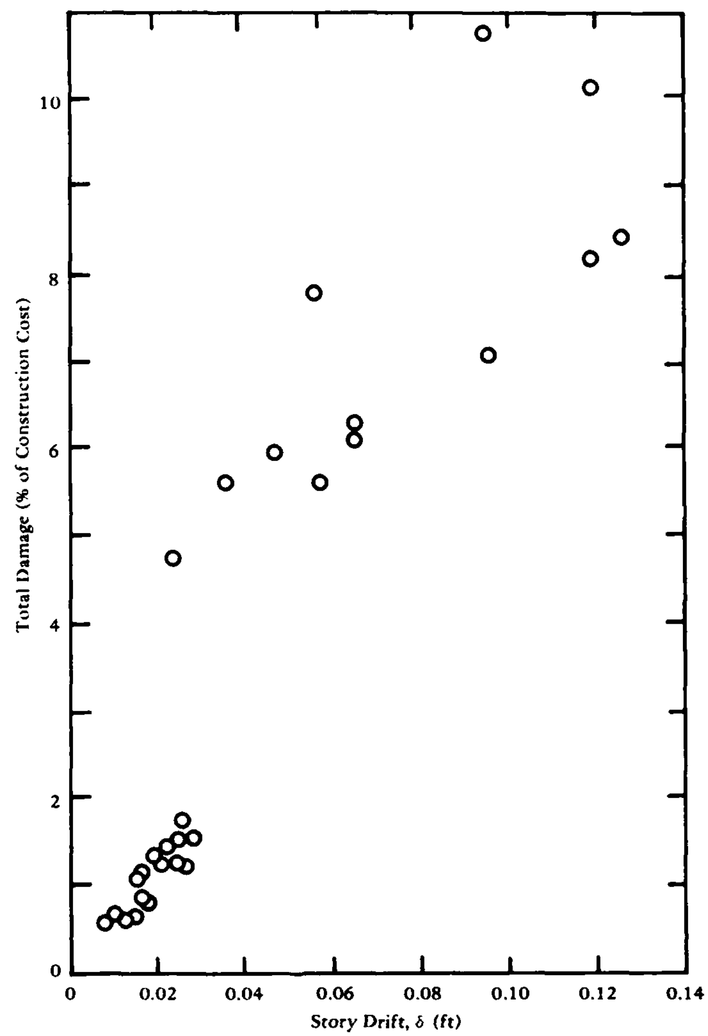


Figure 5. Total damage versus story drift (from Ref 19).

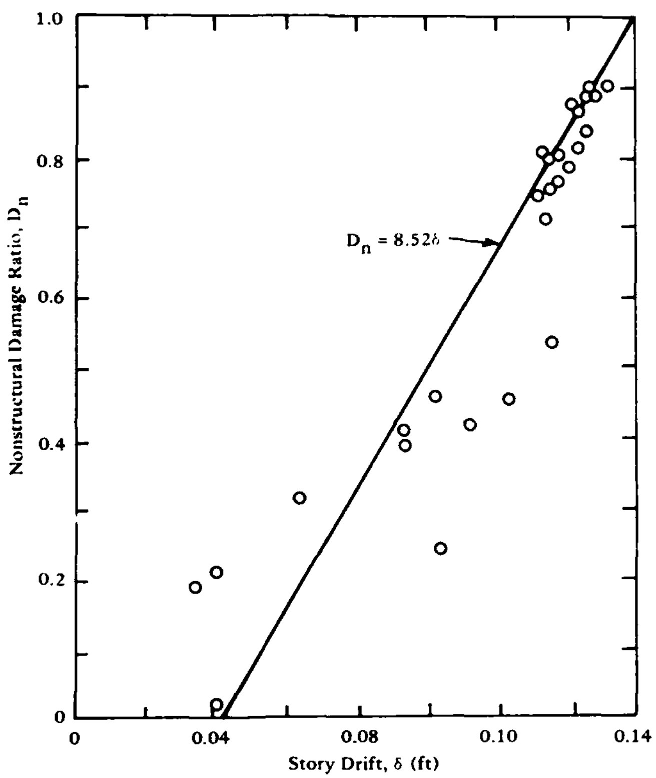


Figure 6. Damage ratio versus story drift (from Ref 19).

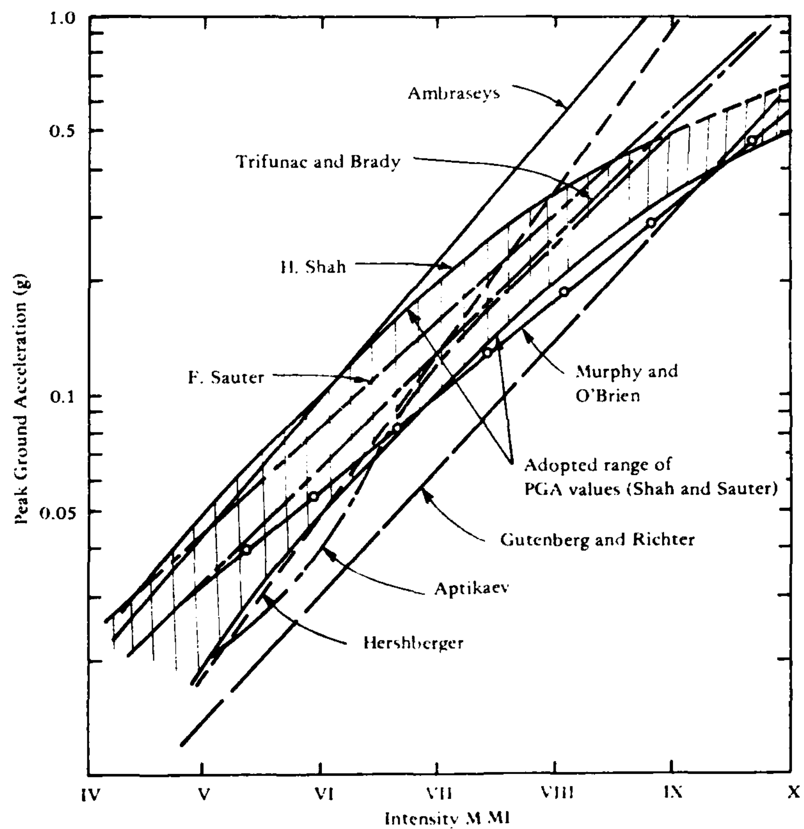


Figure 7. Existing relationships and proposed ranges (Ref 18).

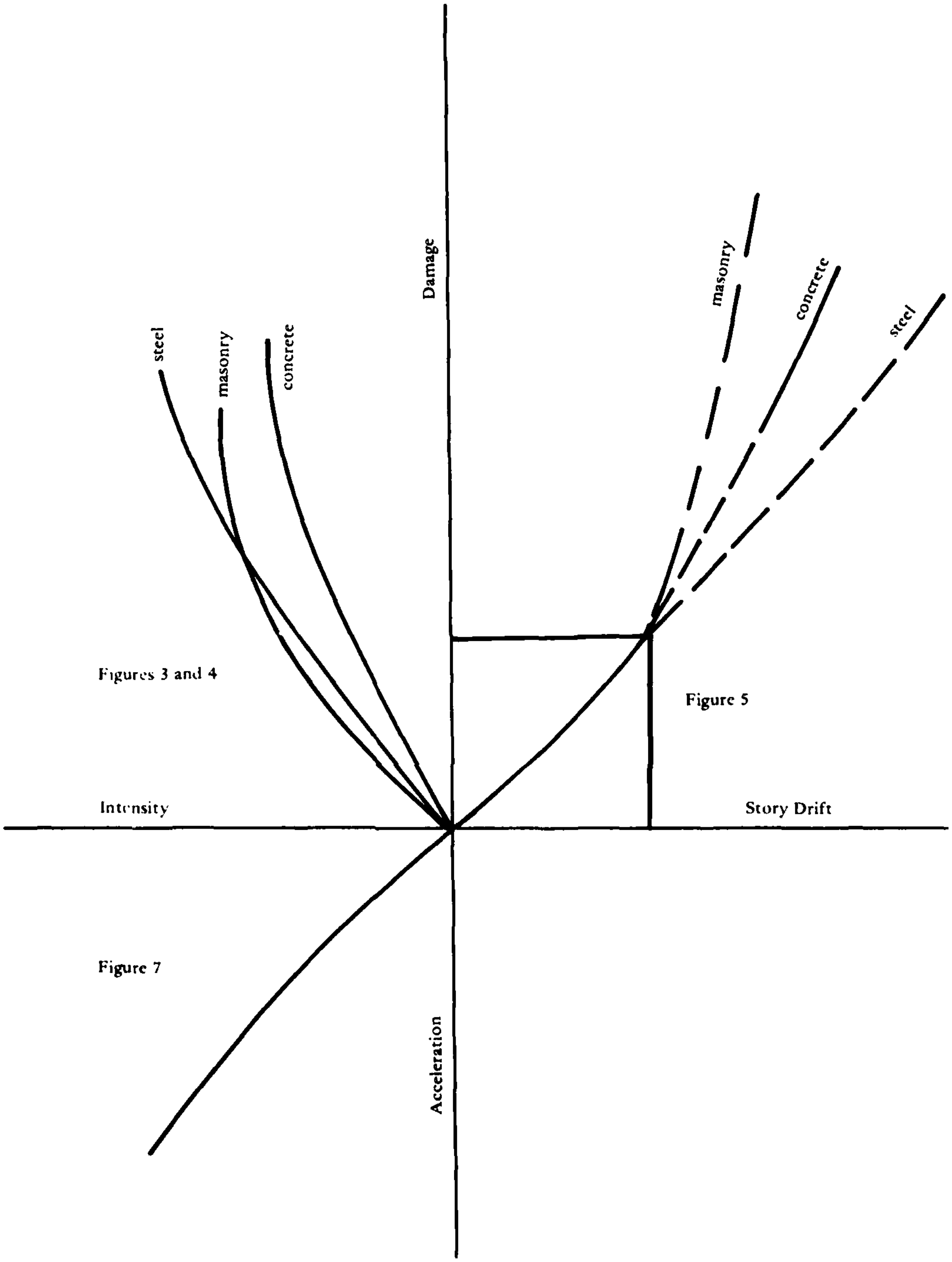


Figure 8. Concept of relating acceleration to story drift and damage.

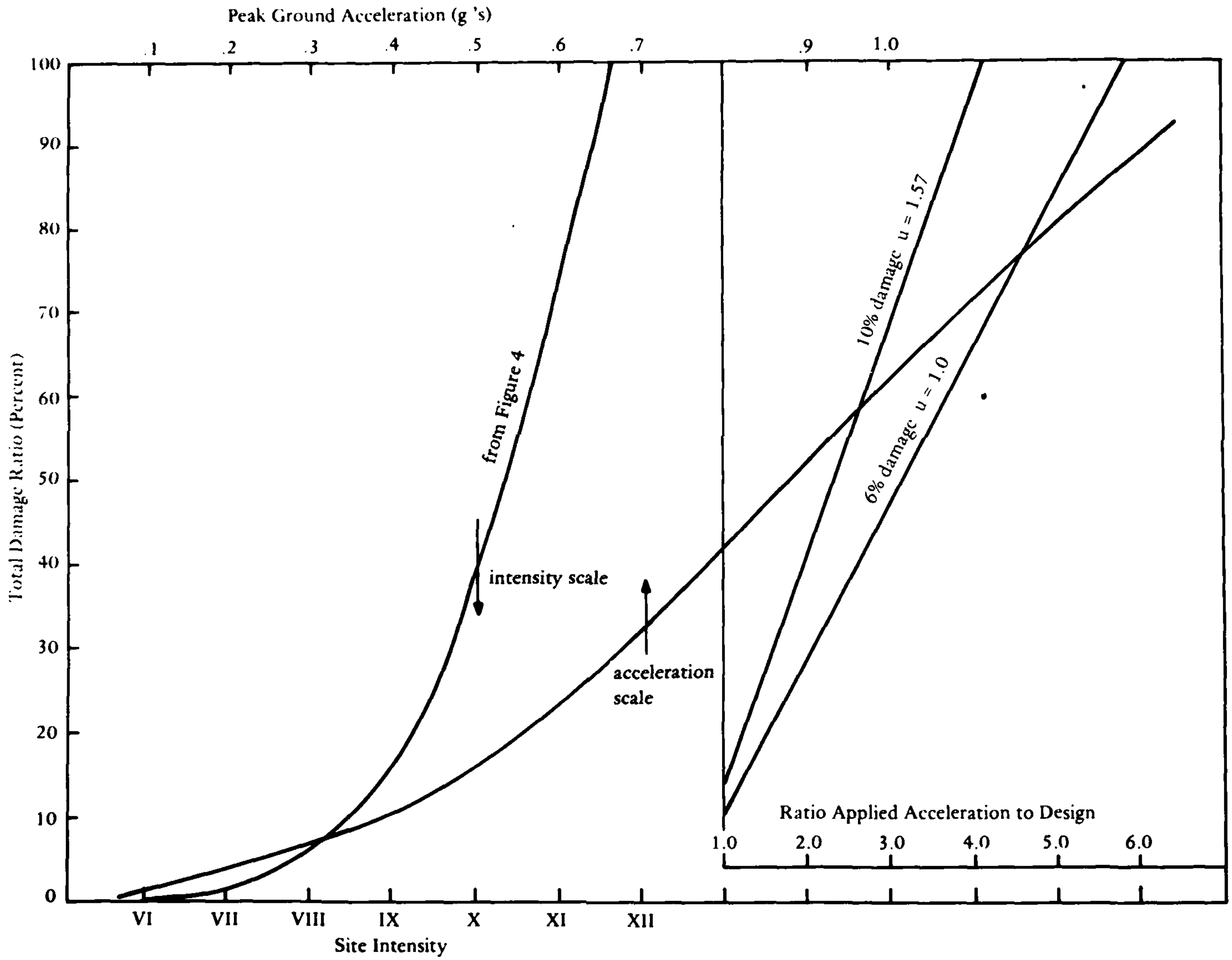


Figure 9. Reinforced masonry with seismic design.

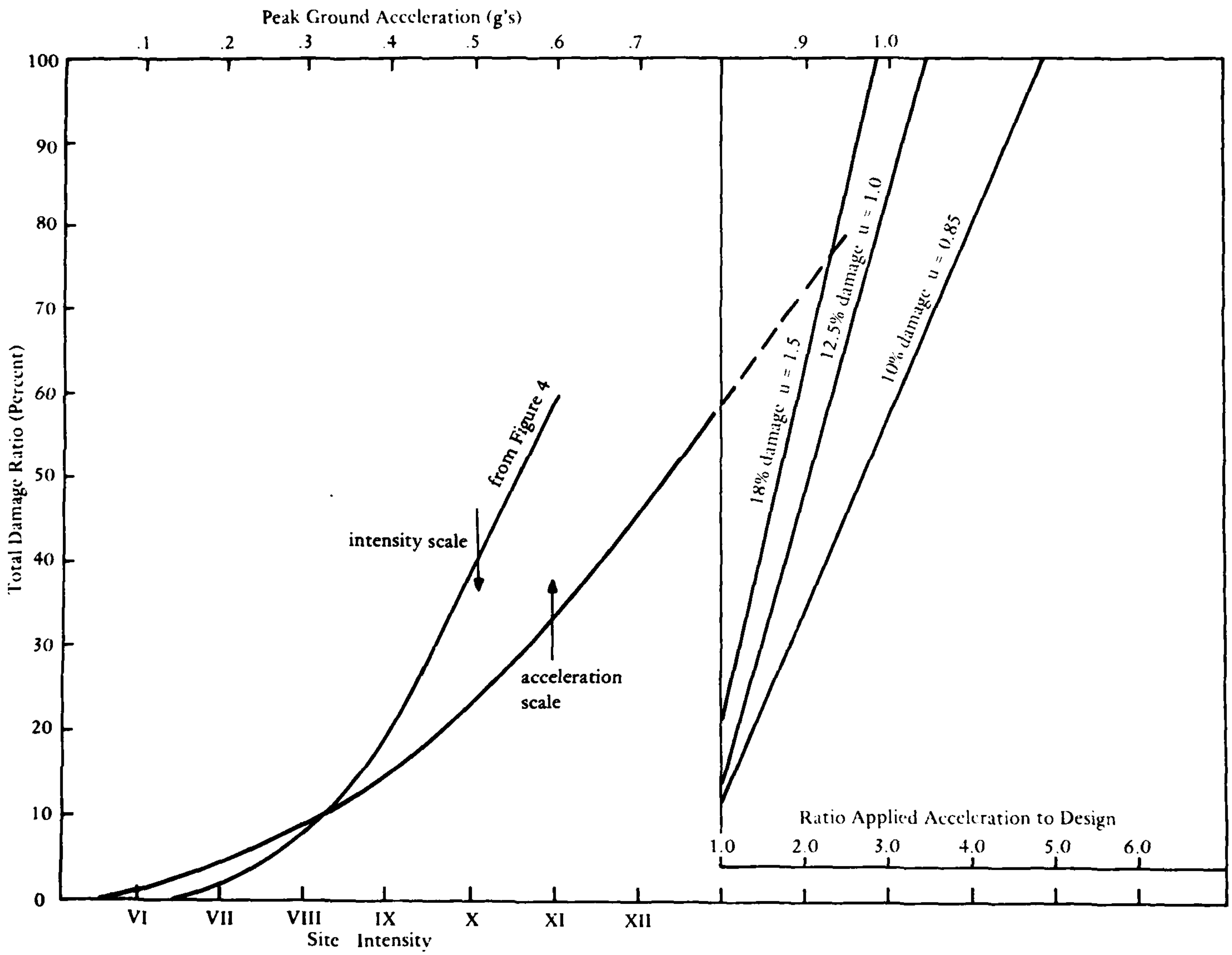


Figure 10. Steel frame building with seismic design.

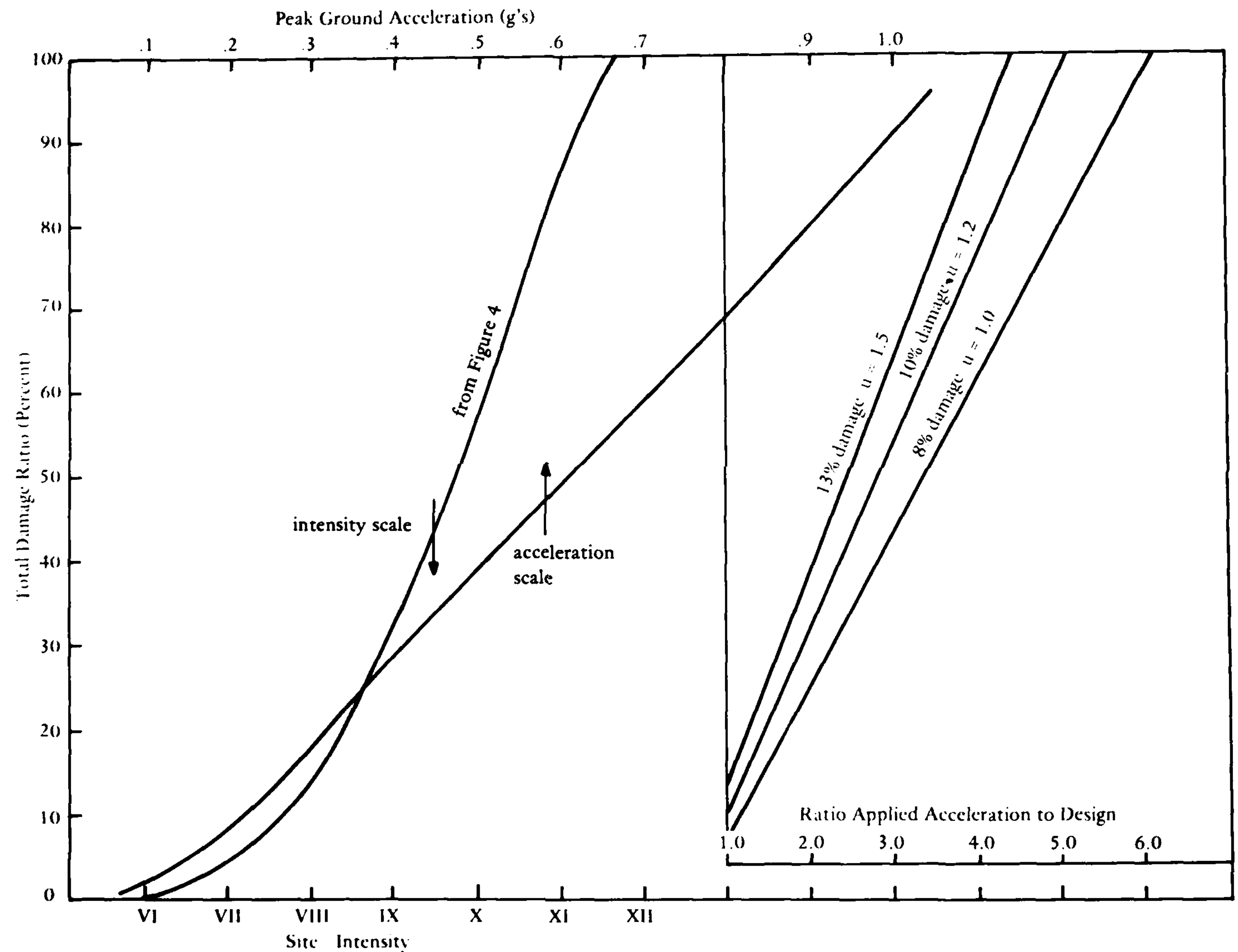


Figure 11. Concrete frame building with seismic design.

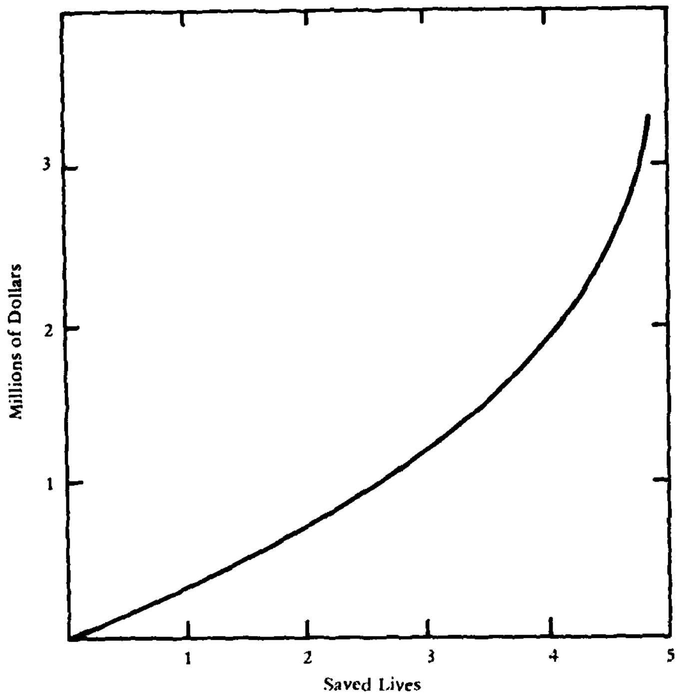
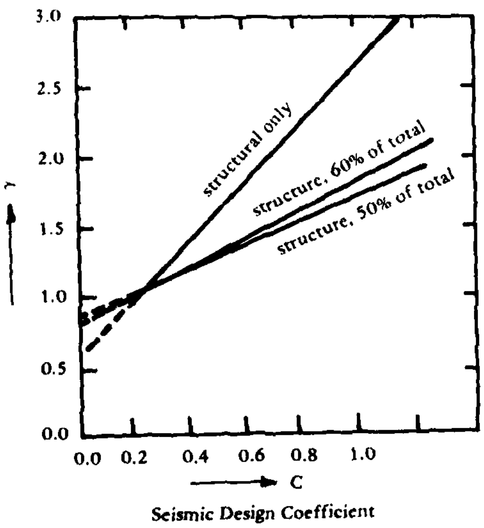
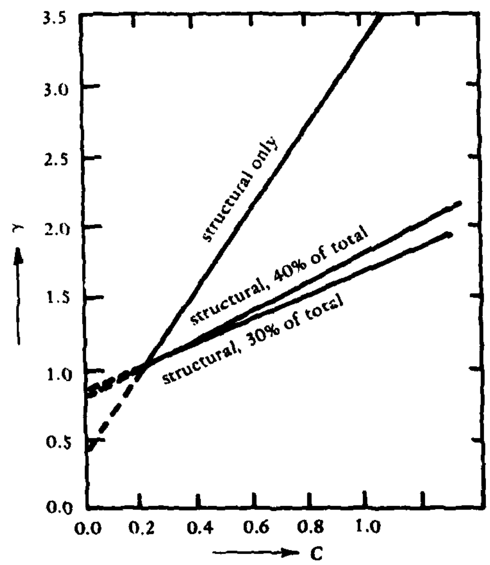


Figure 12. Cost to save human life (from Ref 20).



(a) Steel frame.



(b) Reinforced concrete (N=5, n=∞ in both directions, no aseismic shear walls)

Figure 13. Cost ratio of five-story buildings (Ref 22).

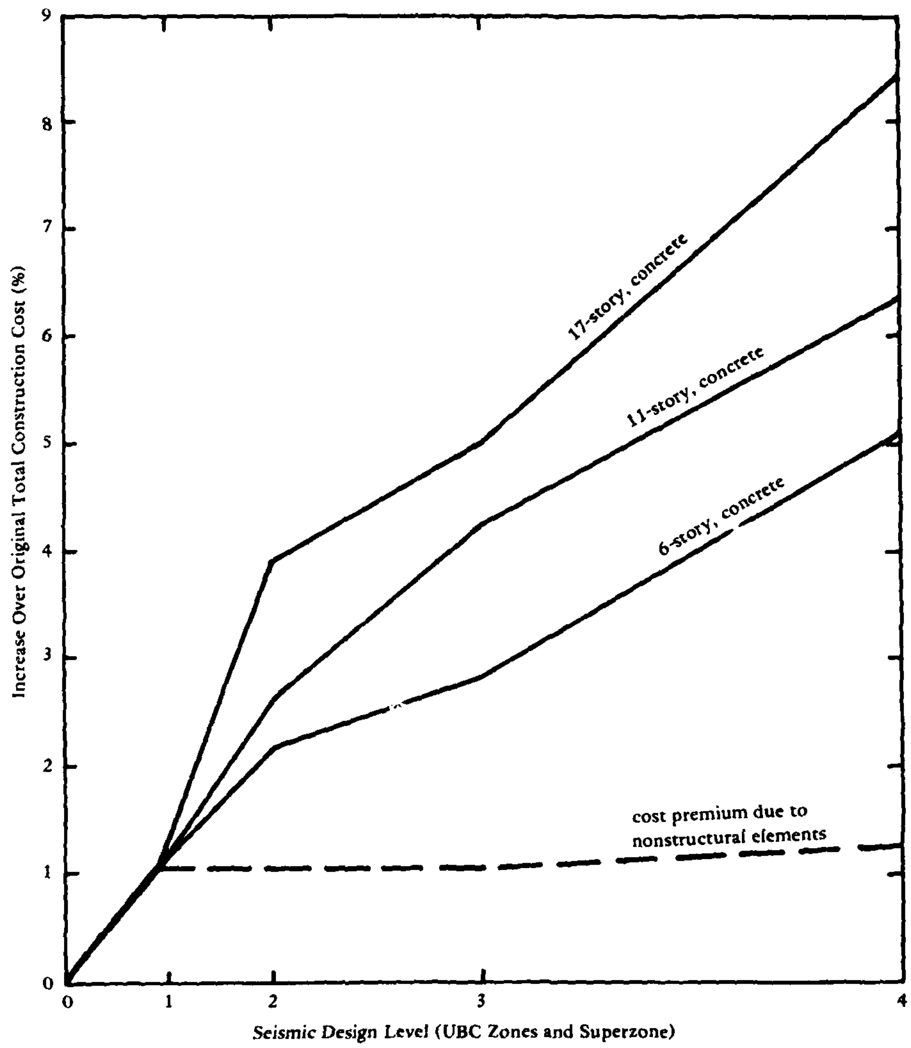


Figure 14. Cost increase (© "Seismic design decision analysis," R. V. Whitman et al., ST5, ASCE, May 1975; Ref 23).

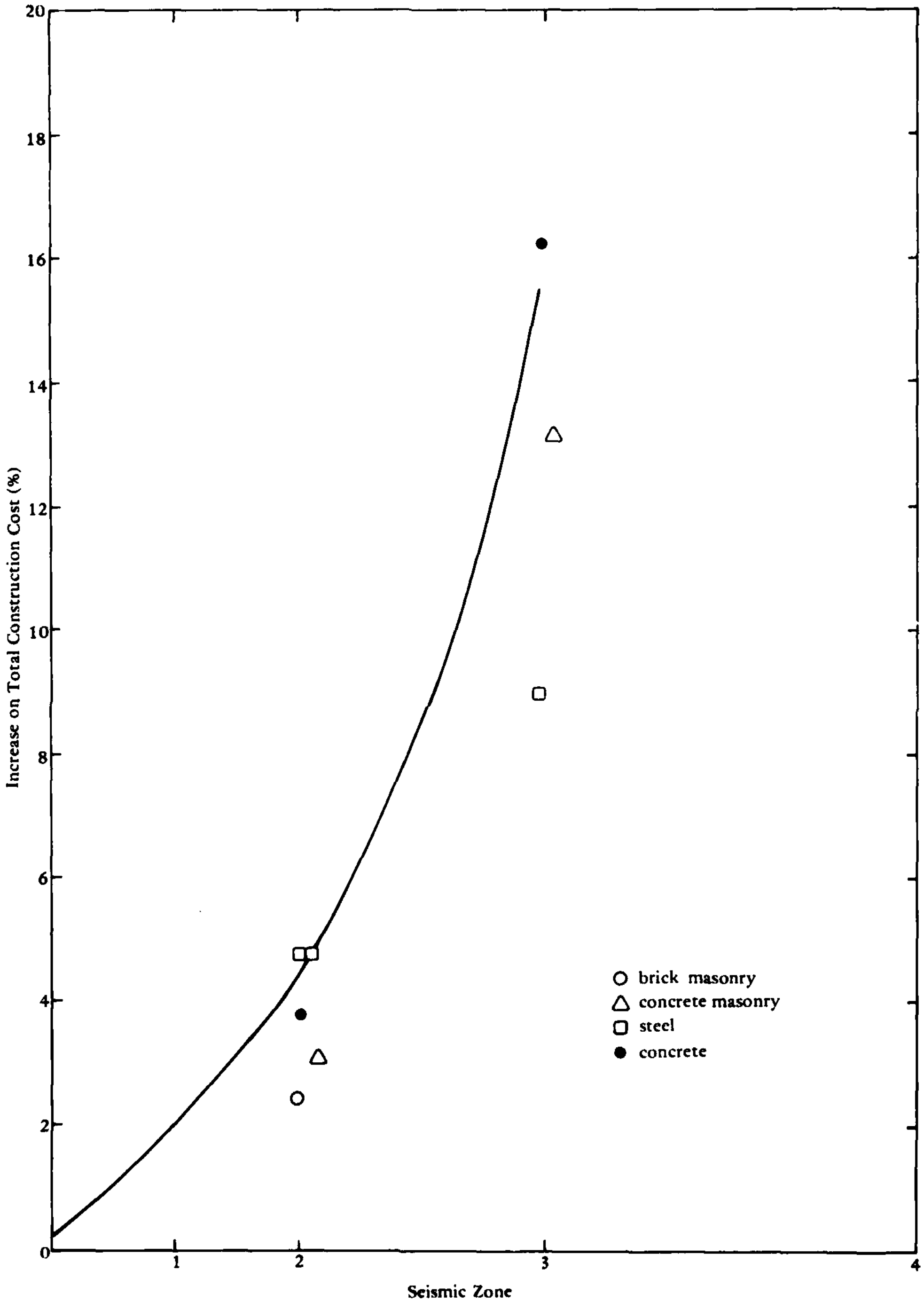
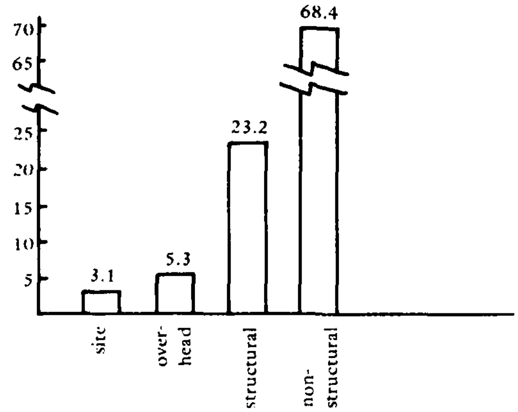
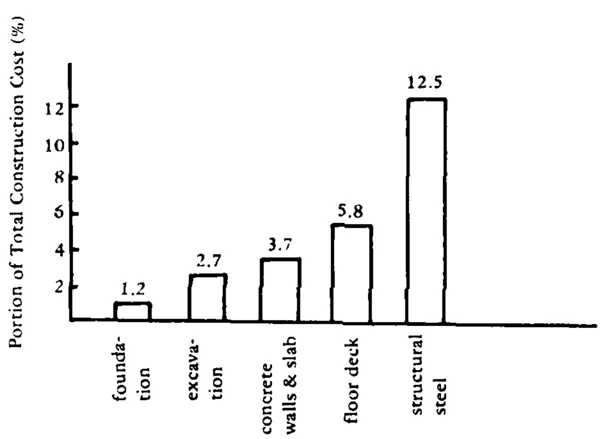


Figure 15. Cost increase based on HUD data.



(a) Original construction costs.



(b) Structural costs.

(c) Nonstructural costs.

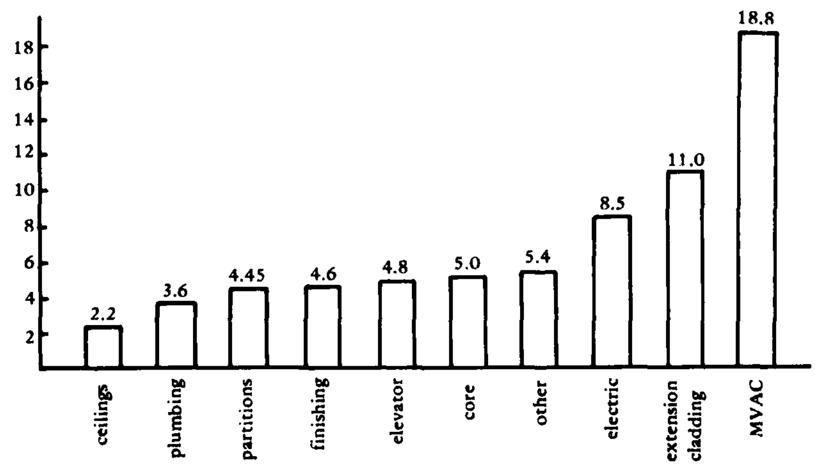
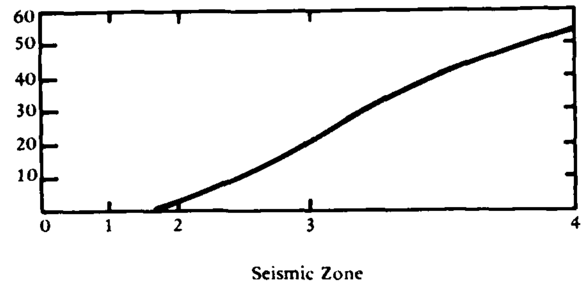
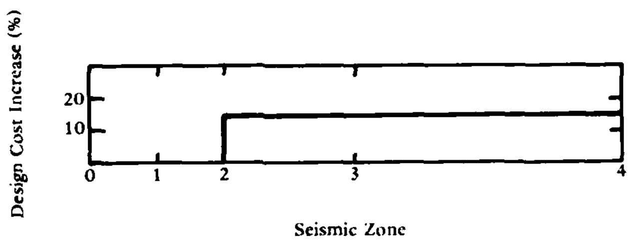


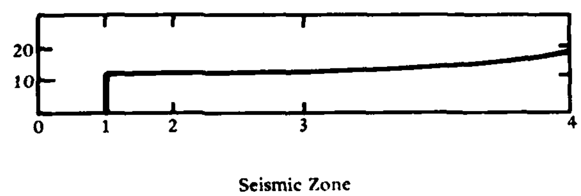
Figure 16. Breakdown of the original construction costs of building (Ref 22).



(a) Steel frame.

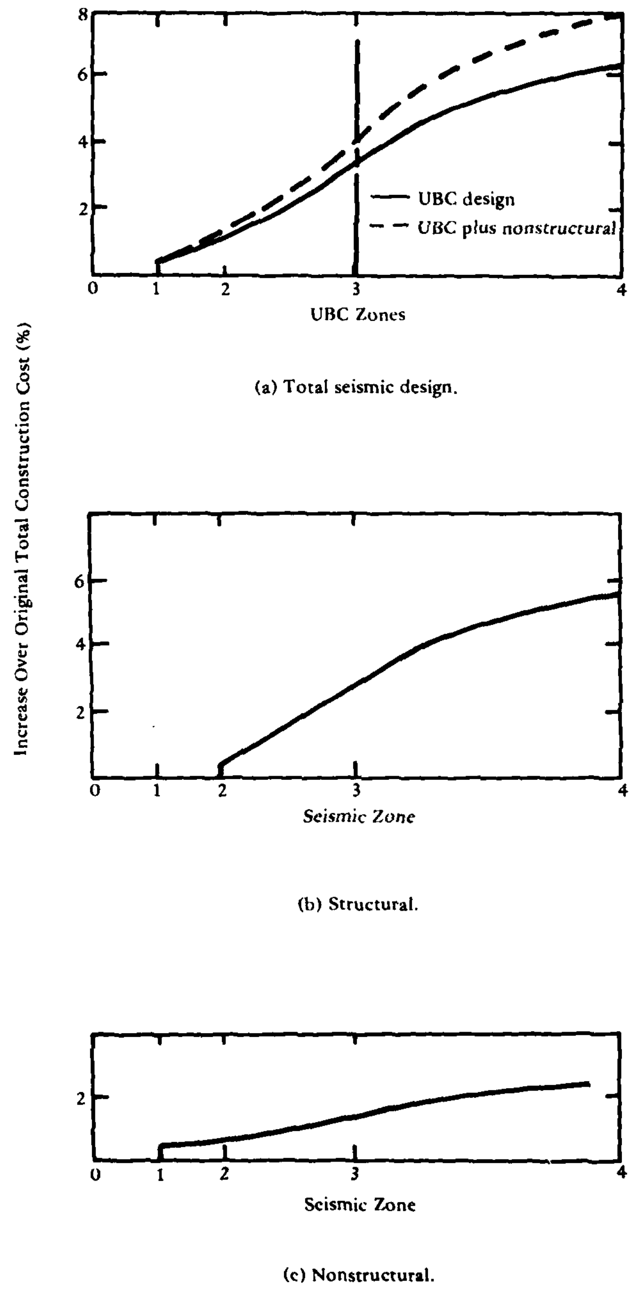


(b) Foundation.



(c) Core wall.

Figure 17. Cost increase for seismic design (Ref 22).



(a) Total seismic design.

(b) Structural.

(c) Nonstructural.

Figure 18. Cost increase for seismic design case study building (Ref 22).

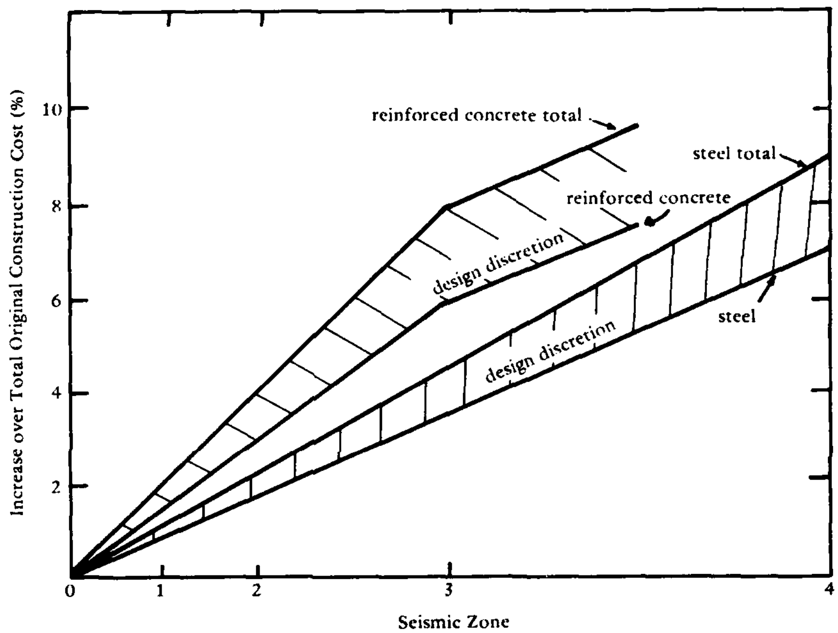


Figure 19. Maximum probable seismic cost increase (Ref 22).

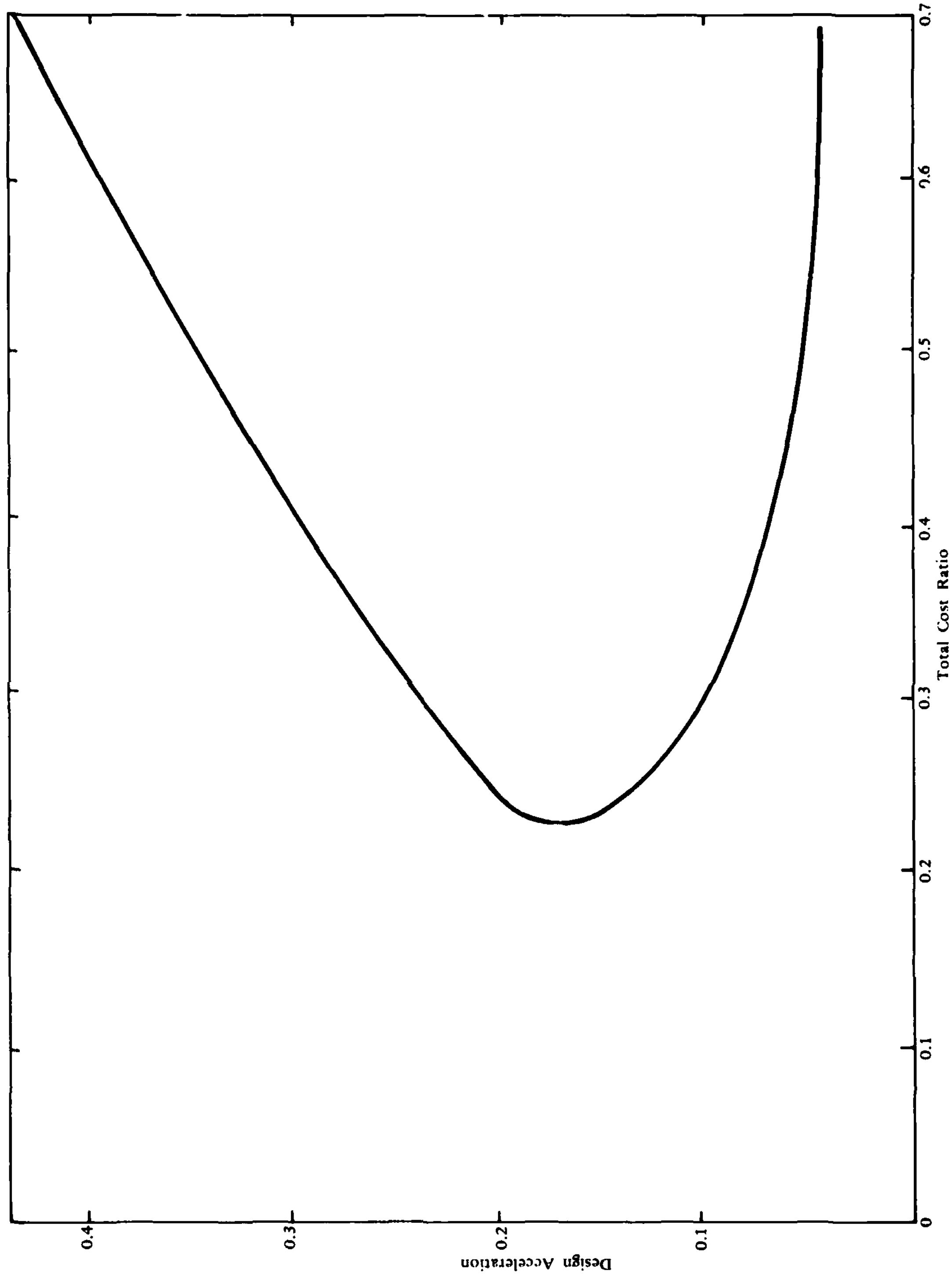


Figure 20. Cost ratio and design acceleration.

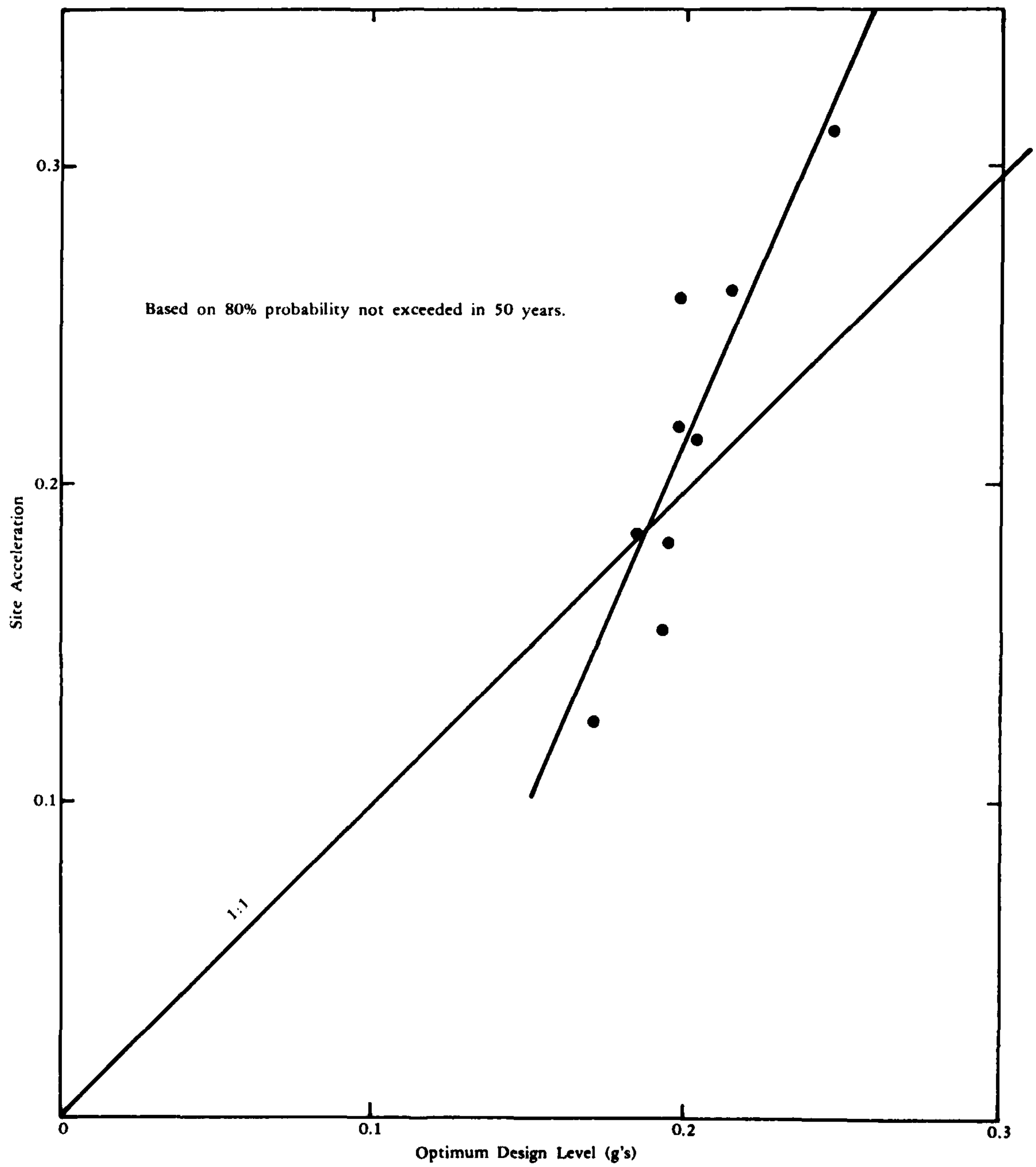


Figure 21. Site acceleration not discounting future damage.

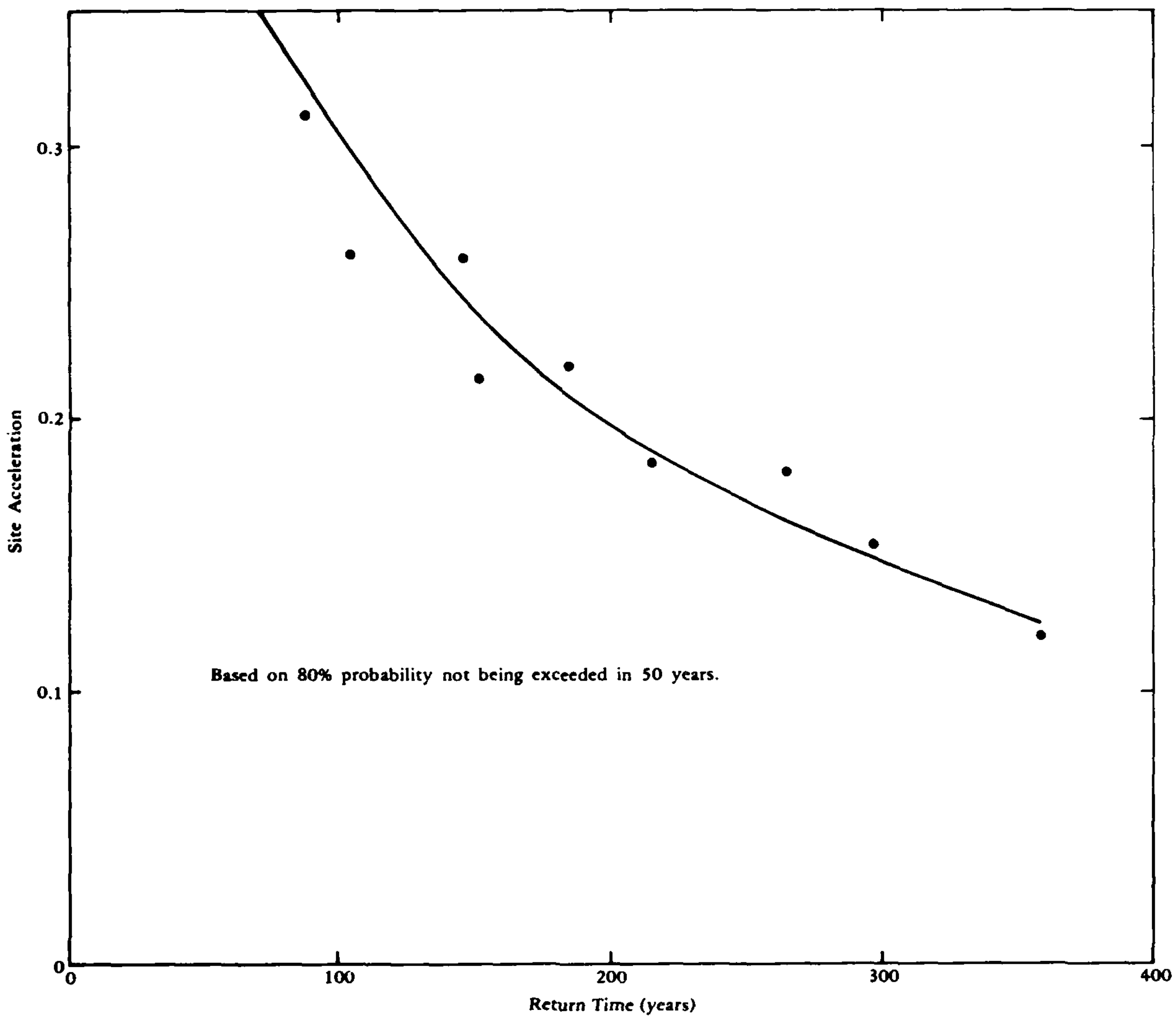


Figure 22. Least cost design acceleration return time (years).

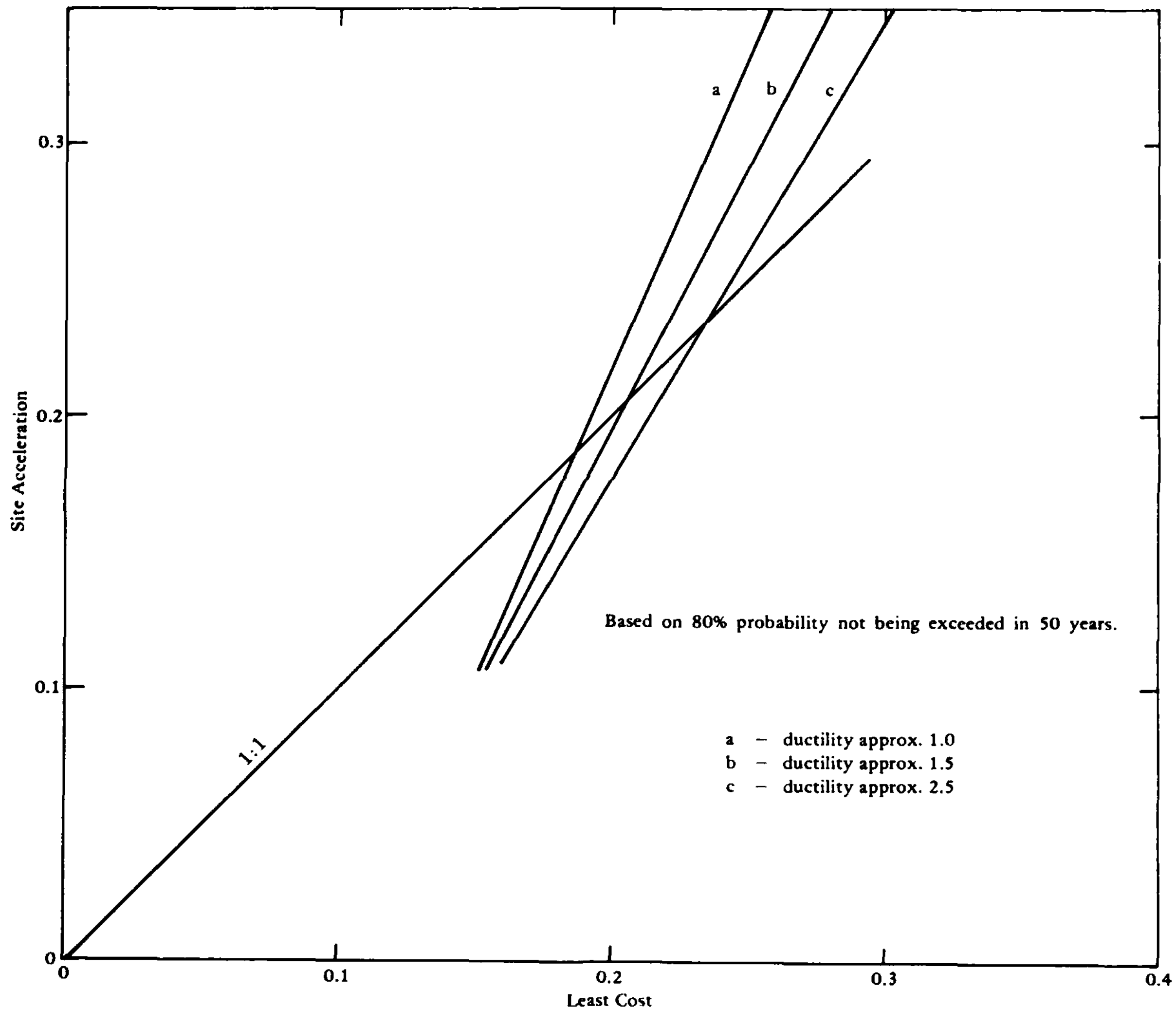


Figure 23. Least cost design acceleration level (years).

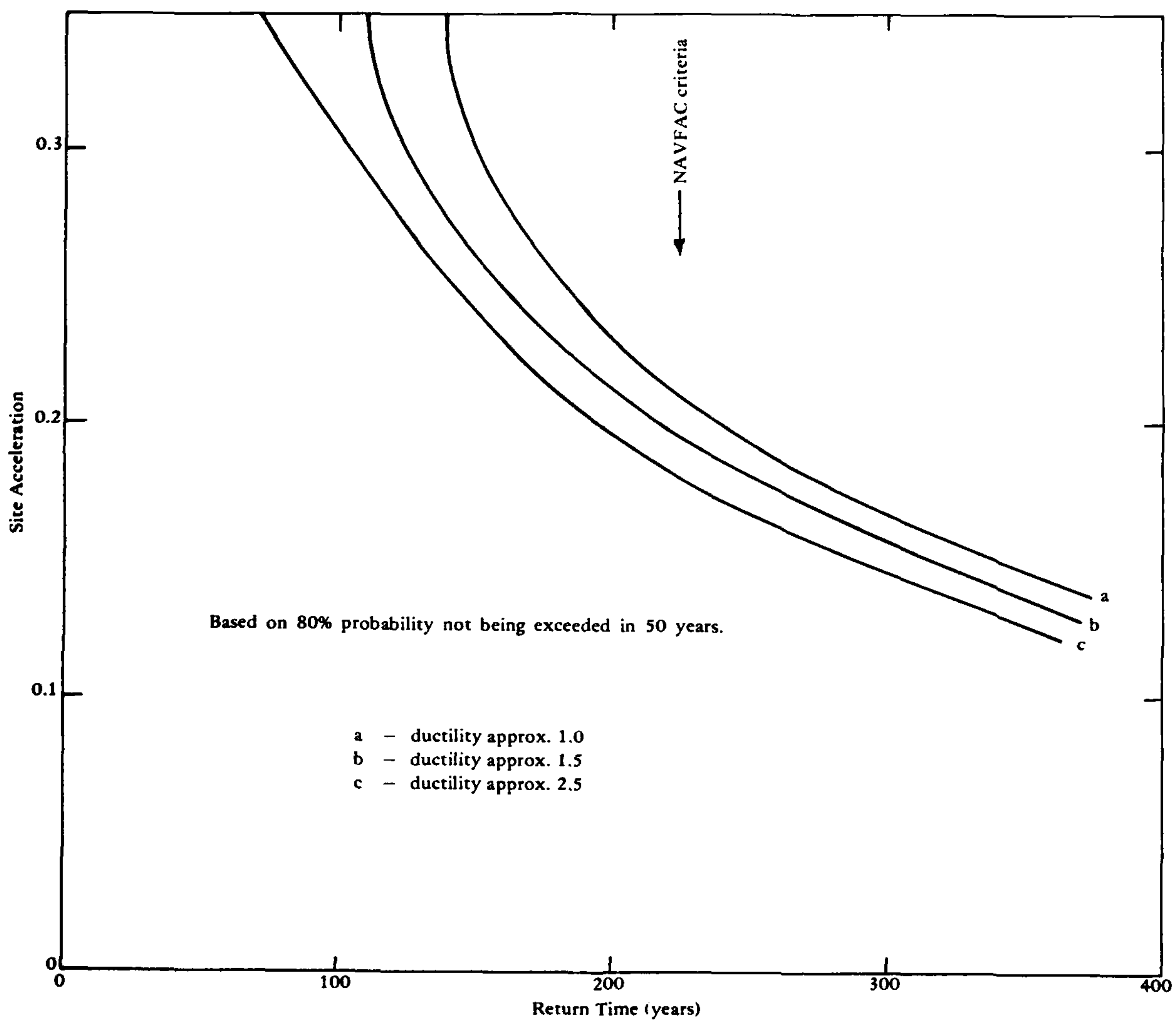


Figure 24. Least cost design acceleration return time (years).

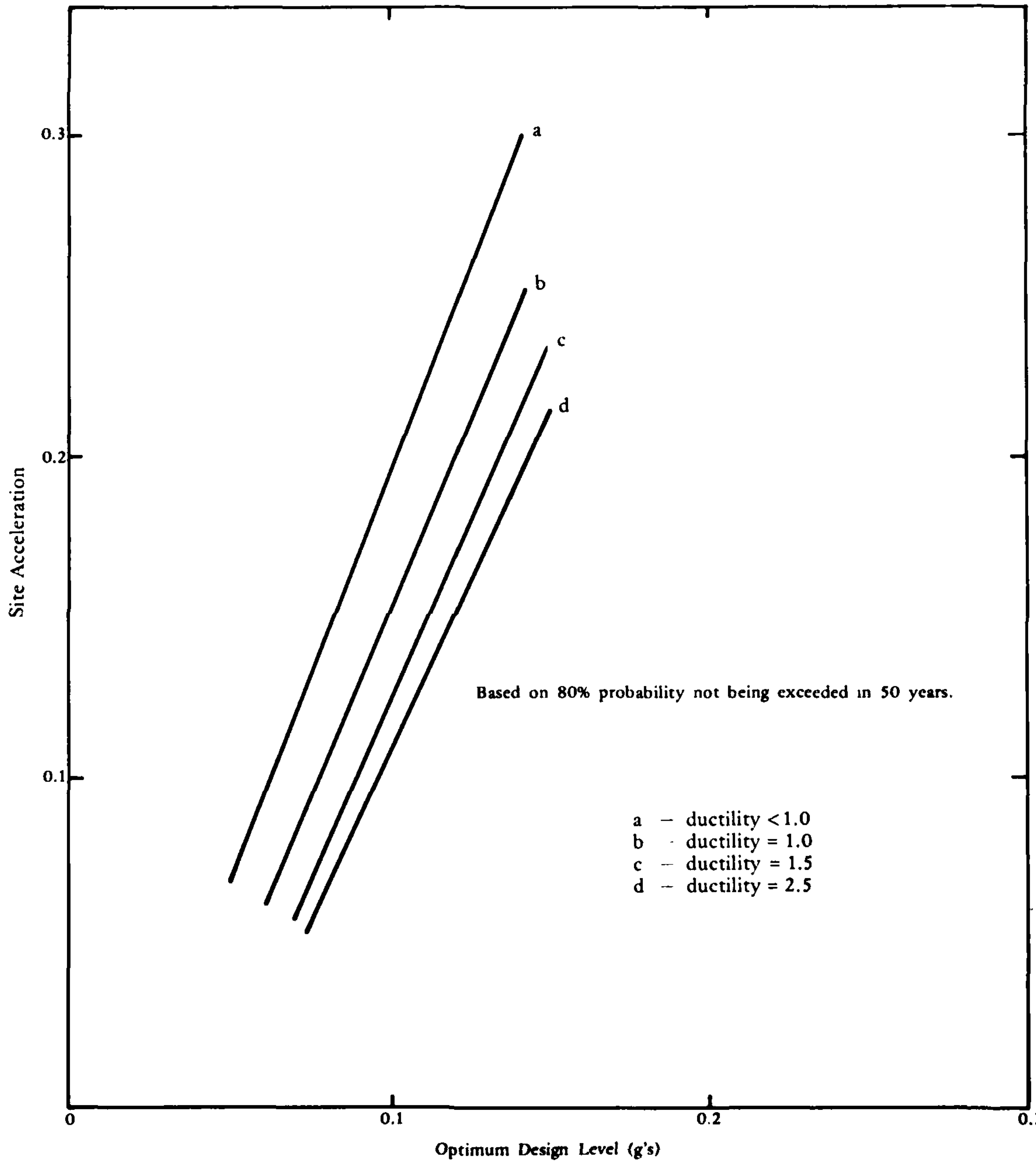


Figure 25. Design acceleration discounting future damage.

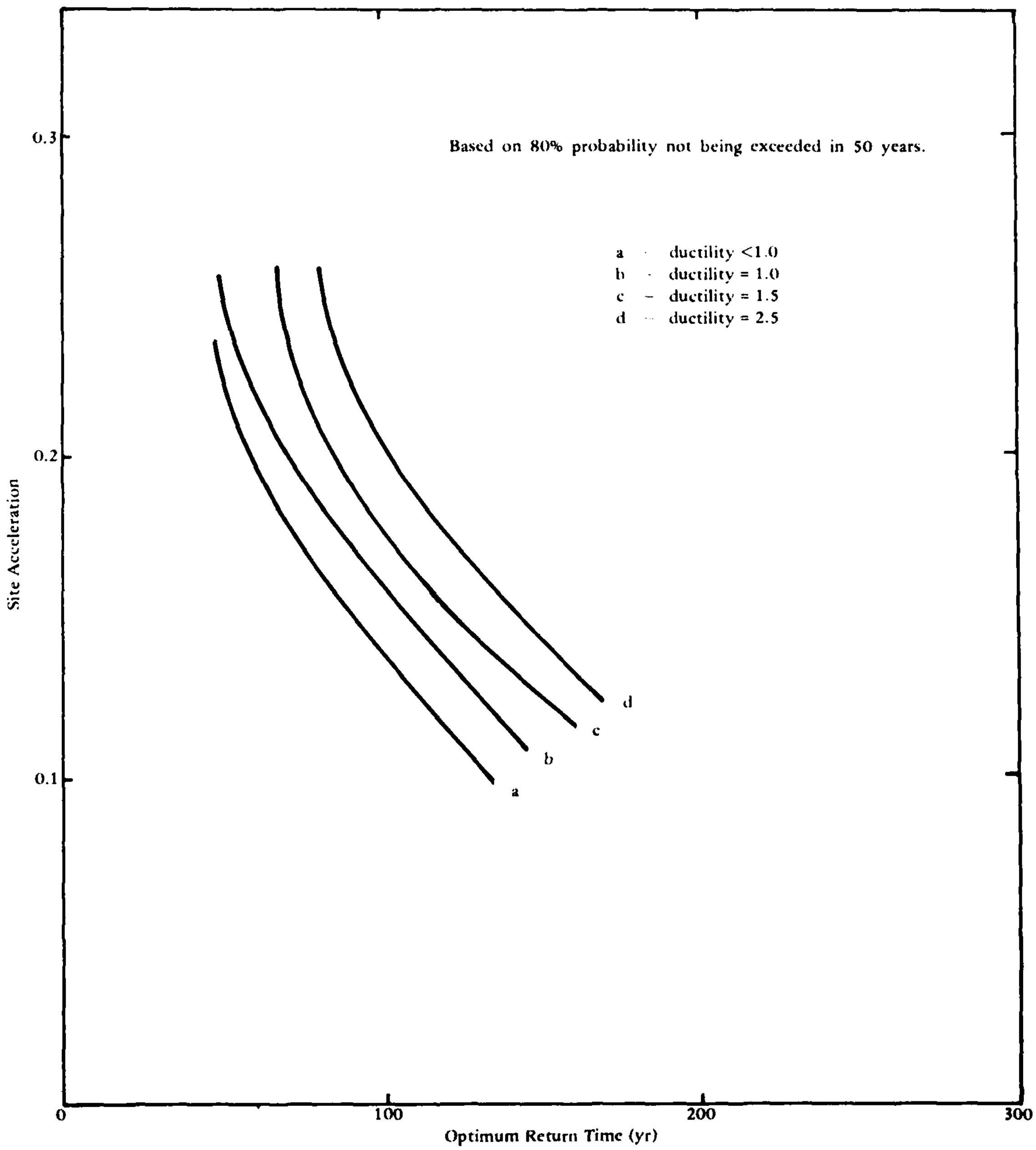


Figure 26. Design return time discounting future damage.

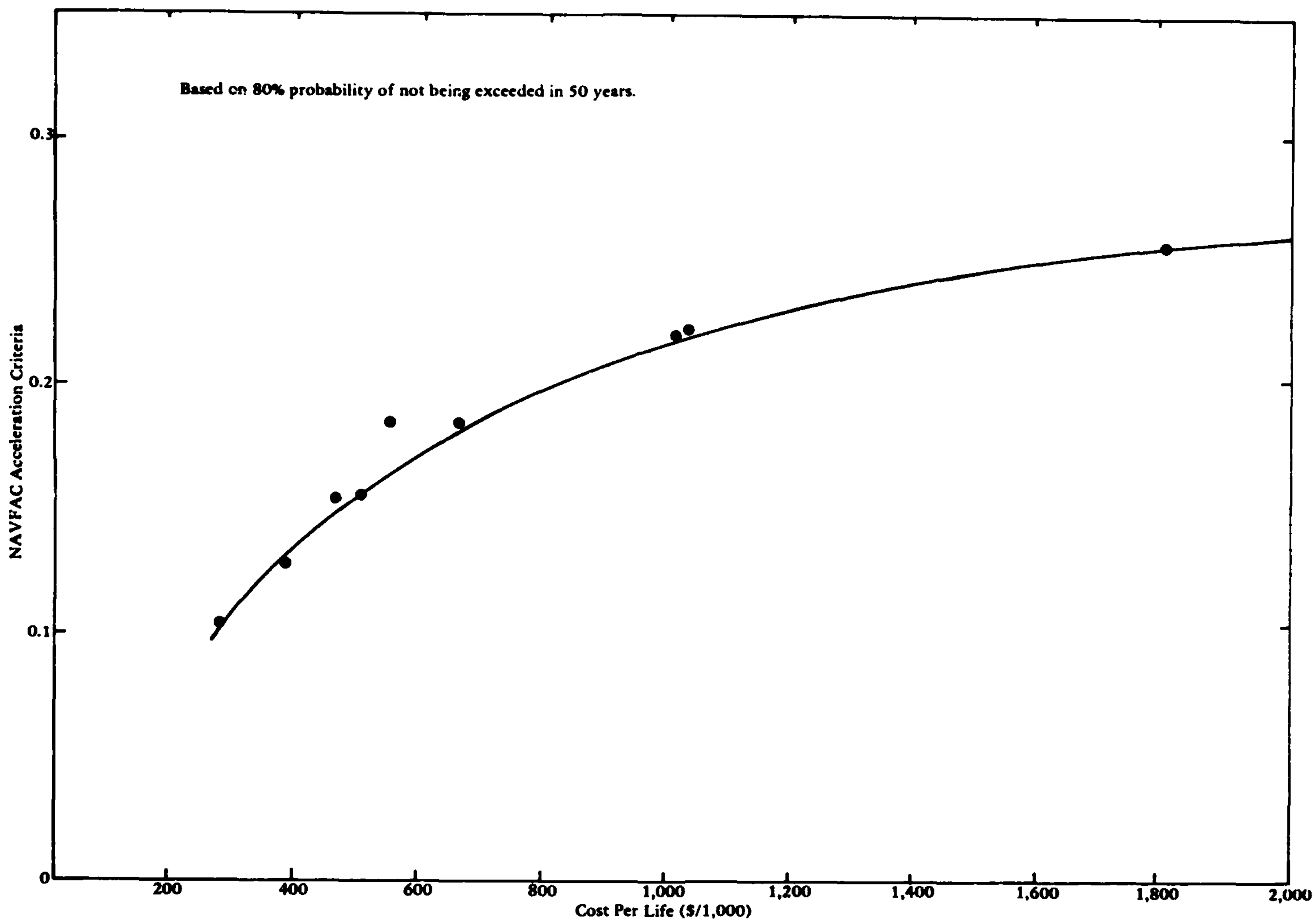


Figure 27. Cost per life.

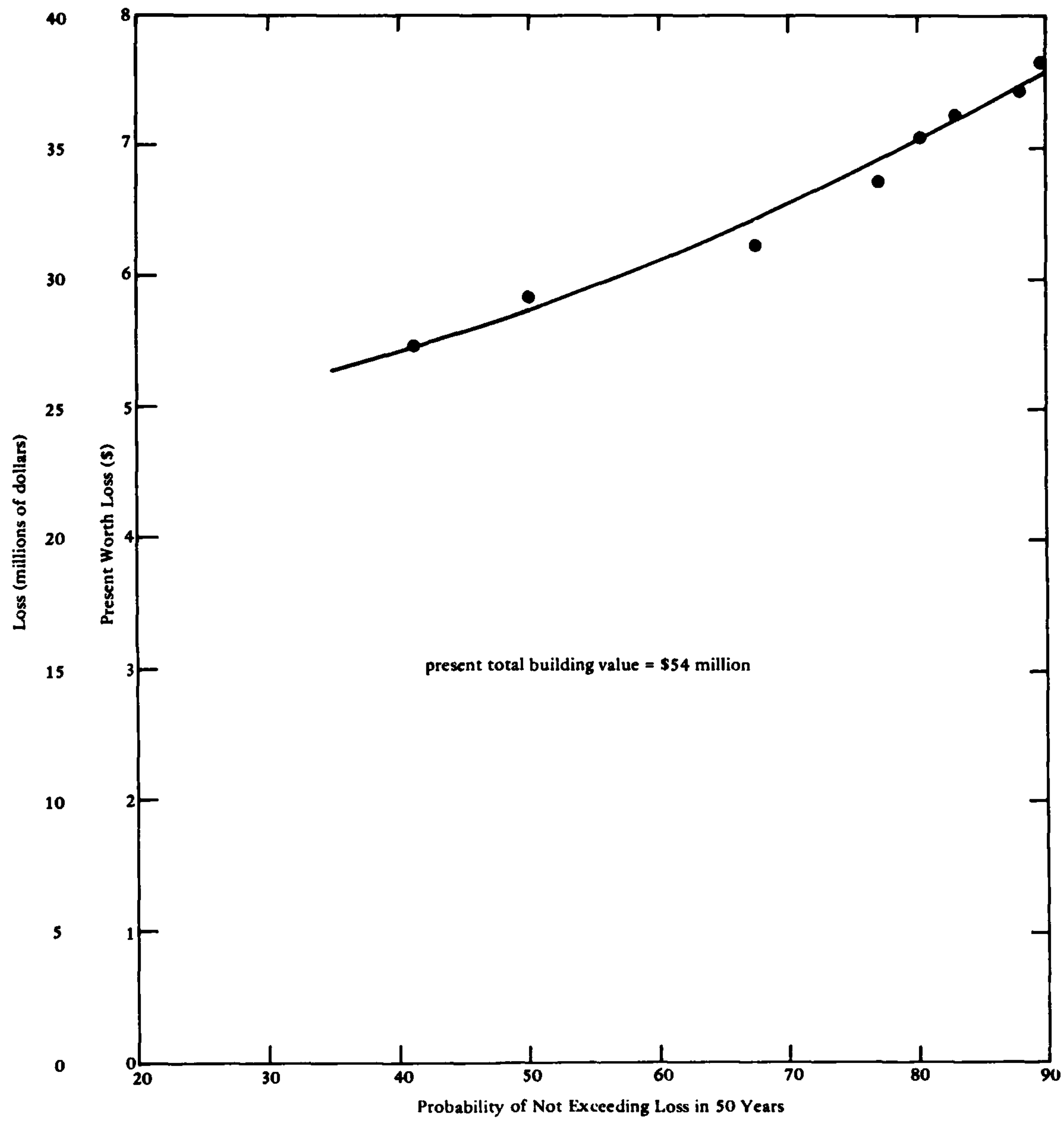


Figure 28. Probability of loss.

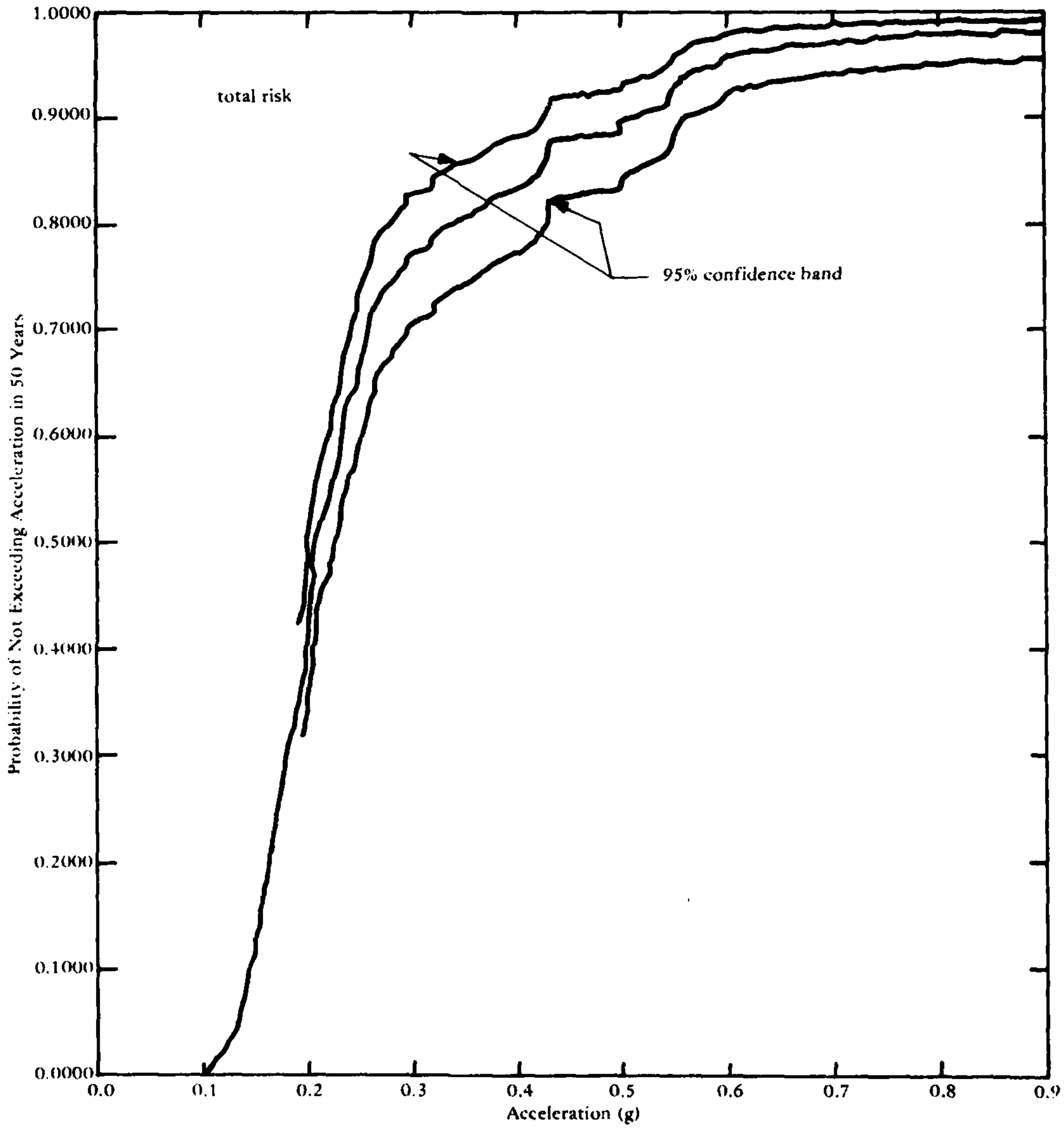


Figure 29. Total probability of not exceeding acceleration at the site with all faults contributing.

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